TABLE OF CONTENTS

14.1 GENERAL

(1) Introduction

Retaining structures are used to hold back soil where an abrupt change in ground elevation is required to accommodate a transportation facility. In some cases right of way restrictions or the presence of existing structures or roadways necessitates the use of earth retaining structures. In other cases the soil may be constructed at a natural stable slope or at a steeper slope by utilizing some type of soil reinforcement. In the past when an earth retaining structure was required a cast-in-place concrete cantilever retaining wall, either on piles or spread footings, was designed and built. Today with the advancements in mechanically stabilized earth (MSE) technology and the development of many proprietary wall systems by the private sector there are many alternatives to cast-in-place walls that are cost effective and are also aesthetically pleasing.

The advent of proprietary wall systems has prompted the Wisconsin Department of Transportation (WisDOT) to evaluate the existing process of designing and letting to contract retaining walls. Many questions arose during the evaluation process but one of the chief issues was how can designs of proprietary wall systems be controlled to produce end products which are equivalent both functionally and aesthetically. The designs of proprietary wall systems are produced by the wall supplier and in order to ensure functional equality among the numerous systems it became necessary for the WisDOT to develop design procedures and criteria for each type of wall system (both proprietary and non proprietary) allowed on WisDOT projects. These design procedures and criteria and usage restrictions are given in the Section of this chapter describing the various wall systems allowed on WisDOT projects.

Aesthetic equivalence being somewhat intangible and project specific is more difficult to determine than functional equivalence. The burden of establishing aesthetic equivalence falls on the designer who is most familiar with the overall project concept and has received public opinion input from public hearings or other forms of communicating with citizens who have an interest in a specific project.

All proprietary wall systems must be pre-approved by the WisDOT prior to being considered or used on WisDOT projects. All systems must be designed in compliance with the design procedure specified in the Bridge Manual. This design procedure may be different than the design procedure normally used by the wall supplier when designing walls for non Wisconsin DOT projects. For proprietary walls and contractor designed non-proprietary walls, the Structures Design Section shall have the responsibility of reviewing the structural aspects of the design and construction plans provided to the District project engineer by the contractor. Note that the structural design of proprietary wall systems is the responsibility of the wall supplier (vendor). After the award of the contract the contractor must submit complete plans and shop drawings to the District project engineer. The contractor must provide the State within a specified time period the name of the vendor who will be supplying a proprietary wall system.

(2) Wall Contract Process

WisDOT has classified all walls (both proprietary and non-proprietary) into eleven wall systems. All proprietary systems must be pre-approved prior to being considered for use on WisDOT projects. The eleven wall systems that are considered for WisDOT projects include the following:

1. Cast-In-Place Concrete Walls
2. Post and Panel Walls With or Without Soil Anchors
3. Gabion Walls
4. Sheet Piling Walls With and Without Soil Anchors
5. Modular Block Gravity Walls
6. MSE Walls with Precast Concrete Panel Facings
7. MSE Walls with Modular Block Facings
8. Modular Concrete Bin or Crib Walls
9. MSE Wire Faced Walls
10. MSE Walls with C.I.P. Facing
11. Stone and Boulder Walls (under development)

Specialty wall systems, combination wall systems, or wall systems not specifically included in the above list may be considered on a case by case basis for individual projects.

Wall system numbers 1, 2, 4 and 11 are non-proprietary. Wall system number 3 can be either proprietary or non-proprietary. Wall system numbers 5, 6, 7, 8, 9 and 10 are typically proprietary wall systems.

If a wall is required, the designer must determine which wall system or systems are suitable for a given wall location. In some locations all wall systems may be suitable, whereas in other locations some wall systems may not be suitable. Section 14.2 gives items to be considered when selecting suitable retaining wall systems for a given site. Section 14.3 also contains restrictions on the use of some wall systems. Information on aesthetic qualities and special finishes and colors of proprietary systems is available from the manufacturers. Designers are encouraged to contact the Structural Development Unit (608-266-8494) if they have any questions about the material presented in the Bridge Manual or the suitability of wall systems for their project.

The step by step process required to select a suitable wall system or systems for a given wall location is as follows:

Step 1: Investigate Alternatives

Investigate alternatives to walls such as extra right of way, slope stabilization, soil reinforced slopes, or alignment shifts in cooperation with the involved geotechnical engineer. These alternatives may be less expensive than designing/constructing/maintaining a retaining wall.

Step 2: Select Suitable Wall Systems

Considering the items presented in Section 14.2 "Wall Selection Criteria" of the Bridge Manual and usage restrictions stated in Section 14.3 (E) and (F), a list of suitable wall systems should be prepared.

Step 3: Specifying Wall System or Systems

The designer should specify only one wall system, either proprietary or on-proprietary, from the list of suitable systems. If the designer elects to specify a proprietary wall system and two or more proprietary systems are interchangeable then more than one proprietary system should be specified. For example, a closed face modular concrete bin wall can usually be substituted for a MSE wall with precast concrete panel facings since both are similar in appearance and function. A closed face modular concrete bin wall can be used for bridge abutments. Cost data on wall systems is available from the Structural Development Unit (608-266-8494).

Step 4: Geotechnical Analysis

Wall height is the distance from the top of leveling pad to the bottom of the wall cap. If the typical wall height is greater than 5.5 feet then the proposed wall system or systems is submitted to the involved geotechnical engineer for a geotechnical investigation. Note that the maximum wall height may be greater than the typical wall height.

The designer must apply good judgment when determining if a site should be submitted for a geotechnical investigation. Walls with live loads and/or backslopes produce higher horizontal pressures and vertical soil pressures than equal height walls without live loads and/or backslopes. For example, an MSE wall with a height of 5.5 feet, liveload, 1:2 backslope, and an angle of internal friction of 30° for retained soil produces a soil bearing stress of 1560 psf. The same wall without liveload and backslope produces a soil bearing stress of only 770 psf. The WisDOT Geotechnical Section can be reached at 608-246-7940 to provide cost estimates of a geotechnical investigation and additional information or assistance.

The geotechnical investigation will generally consist of a subsurface boring program and will examine both the retained soil and foundation soil to report the following required items. Additional investigation or testing should be conducted as directed by the Engineer.

1. Angles of internal friction
2. Cohesion
3. Unit weight
4. Ultimate or allowable bearing capacity
5. Settlement
6. Water table location
7. Site geometry
8. Gradation Analysis & Atterberg limits

Additional items that may be required include:

9. Active and passive earth pressures
10. Pile skin friction and end bearing values

An analysis of external stability including resistance to sliding, rotational, global and bearing failures shall be presented for both short-term and long-term conditions. Safety factors for each mode of failure should also be provided.

Soil remediation designs, if necessary for specific wall systems, shall also be included in the report. The report shall also contain an assessment based on the geotechnical analysis of the practicality of the wall system selected by the designer.

Step 5: Determine Contract Letting

After the designer has received the geotechnical report to document the selected wall system or systems the method of contract letting can be determined. The designer has two options:

Option 1: A non-proprietary wall system is specified.

Under Option 1 WisDOT will furnish a completed design for the system.

Option 2: A proprietary wall system (or systems) is specified.

Under Option 2 WisDOT will not furnish a design for any wall system. WisDOT will supply basic layout information including plan, elevation and cross-sectional views and geotechnical information.

Step 6: Prepare Contract Plans

Refer to Section 14.4 for information required on the contract plans for proprietary systems. For non-proprietary systems the contract plans should show the detailed requirements for one wall system.

Step 7: Prepare Contract Special Provisions

The Bureau of Highway Development has Special Provisions for each wall type. The list of proprietary wall suppliers is maintained by Structures Development. This Section is responsible for the Approval Process for earth retaining walls (Appendix A).

The Special Provisions for the selected wall system or systems and the corresponding list or lists of approved suppliers for proprietary systems are inserted into the plan documents by the designer.

Step 8: Submit P.S. and E. (Plans, Specifications and Estimates)

When the plans are completed, submit the project into the P.S. and E. Process. Note that for proprietary walls that do not require subsurface remediation there is one bid item for all wall quantities, square feet of wall measured from top of footing or leveling pad to top of wall. When subsurface remediation is necessary it is included as separate bid items. For non-proprietary walls, separate bid items should be included for each of the individual components of the specific wall type.

Step 9: Preconstruction Review

For proprietary wall systems the contractor must supply the name of the wall system supplier and pertinent construction data to the project manager. This data must be accepted by the Structures Design Section before construction may begin. Refer to the Construction and Materials Manual for specific details.

If a contractor proposes the use of an alternate wall system, the contractor shall provide all plans and details for the alternate system, including any necessary foundation improvements and an external stability analysis. All plans and accompanying details shall be submitted to the Structures Design Section for review. Any cost savings for alternate wall proposals shall be shared by the contractor and WisDOT as stated in the Standard Specifications for Highway and Structure Construction, Section 104.10, Cost Reduction Incentive.

Step 10: Project Monitoring

It is the responsibility of the project manager to ensure that the project is constructed with the accepted contract proposal. Refer to the Construction and Materials Manual for monitoring material certification, construction procedures and material requirements.

The final wall design is based on several soil and site assumptions. It is imperative that the project manager be familiar with the "Site Investigation Report" and ensure that the wall is constructed according to plan and specifications.

(3) Pre-Approval Process

The purpose of the pre-approval process is to ascertain that a particular proprietary wall system has the capability of being designed and built according to the requirements and specifications of WisDOT. Any unique design requirements that may be required for a particular system are also identified during the pre-approval process. A design of a pre-approved system is acceptable for construction only after WisDOT has verified that the design is in accordance with the design procedures and criteria stated in the Bridge Manual.

The test result submittal requirements for pre-approval vary depending on the type of wall system. In addition to design criteria, suppliers must provide materials testing data and certification results for the required tests for durability, etc.

Preapproval for MSE Systems requires submittal of connection test results for the block or panel and the soil reinforcing to be used with that block or panel. Substitutions of other soil reinforcing is not allowed unless connection tests have been submitted prior to contract letting.

The submittal requirements for the pre-approval process and other related information are given in Appendix A.

14.2 WALL SELECTION CRITERIA

The following items should be considered when generating a list of acceptable retaining wall systems for a given site.

1) Project Category

- permanent or temporary wall
  A temporary wall must meet the physical requirements with very little concern for aesthetics.

- living or non-living wall required
  A living wall requires bin or crib walls unless other wall systems are used with terraces.

2) Site Conditions Evaluation

- cut or fill
  This condition needs to be evaluated because some wall types do not work well for some cases. Determine if top down construction is required for a cut.

- soil profile and site geology
  Evaluate the project for variations in wall height and blending the wall to the site.

- foundation conditions and capacity
  The soil foundation must be evaluated for capacity to support the wall system.

- soil remediation required/feasible
  While certain soil conditions may not support certain wall types, it may be economical to use soil remediation to accommodate these wall types.

- ground water table location
  Consider whether ground water will increase lateral soil pressure on the wall or increase the corrosion potential.

- underground utilities and services
  If utilities seriously interfere with soil reinforcement or other wall elements, consider other wall systems.

- other structures adjacent to site
  Determine if adjacent structures may be affected by wall construction such as piledriving or lack of lateral support.

- corrosive environment and effect on structural durability
  Evaluate the ph of soil and its effect on a minimum 75 year design life.

3) Performance Criteria

- height limitations for specific systems
  Check the height limits for the wall systems as well as practical design limits.

- limit of radius of wall on horizontal alignment
  Evaluate wall system to accommodate any radius situation or adjust radius to meet wall system.

- allowable lateral and vertical movements, foundation soil settlements, differential movements
  Determine allowable movements and choose wall systems that will accommodate the movements.

- resistance to scour
  Be sure wall is not susceptible to scour if the condition exists.

- wall is component of an abutment
  Determine wall systems that are compatible with abutment details.

- traffic barrier or roadway pavement supported by wall
  Be sure the wall system will support a traffic barrier if required for the project.

4) Constructability Considerations

The following items should be considered when evaluating the constructability of each wall system for a specific project.

- scheduling
- formwork, temporary shoring
- right of way boundaries
- complicated horizontal and vertical alignment changes
- site accessibility (access of material and equipment for excavation and construction)
- maintaining existing traffic lanes
- vibrations
- noise

5) Environmental Considerations

- minimum environmental damage or disturbance
  Consider the impact of wall systems on environmentally sensitive areas.

6) Cost

- right of way purchase requirements
  Evaluate the cost of additional right of way if it is required to use a given wall system.
- do not base selection on cost of wall systems.

7) Aesthetic Considerations
- see Bridge Manual Section 14.6

8) Mandates by Other Agencies

In certain project locations, other agency mandates may limit the types of wall systems.

9) Requests made by the Public

A Public Interest Finding could dictate the wall system to be used on a specific project.

10. Railing

For safety reasons almost all walls will require a protective railing. The railing will usually be located behind the wall. Determine from the District whether a pedestrian or non-pedestrian railing is required and what aesthetic considerations are needed.

14.3 DESIGN CRITERIA AND INFORMATION

The design procedures and criteria established by WisDOT are generally in conformance with AASHTO "Standard Specifications for Highway Bridges". However, there are some wall system analysis and design procedures which are not specifically covered by AASHTO. For those cases, design procedures have been developed based on standard engineering and/or industry practice. A degree of conservativeness consistent with AASHTO design procedures for other wall types has been included in these procedures.

| The maximum value of the angle of internal friction of the wall backfill material in the
| reinforced zone shall be assumed to be 30 degrees without certified test values. Show the
| value used on the wall plans.

14.3(A) CAST-IN-PLACE CONCRETE CANTILEVER WALLS

A cantilever wall consists of a base slab or footing from which a vertical wall or stem extends upward. Reinforcement is provided in both members to supply resistance to bending. Cantilever walls can be founded on spread footings or on piles. Pertinent soils information on loading conditions, foundation considerations, consolidation potential, and external stability is included in the geotechnical report.

Cantilever walls with heights in excess of 28 feet are more economically constructed when provided with brackets to assist strengthening the junction of the stem and base slab. Counterforts, which are behind the stem and act in tension are normally utilized. For walls on spread footings the front face of the wall should be placed approximately in line with the resultant of the soil pressure distribution. If the resultant is placed at the 1/3 point, the footing length is about 1/2 the height and the soil pressure distribution is triangular.

(1) Design Procedure for Cast-in-Place Concrete Cantilever Walls

Walls shall be designed in accordance with the AASHTO "Standard Specifications for Highway Bridges" except as noted in this section.

The active earth pressure coefficient can be determined from the following formula:

$$Ka = \text{COS } B * (\text{COS } B - X)/(\text{COS } B + X)$$

where B = slope angle of soil behind wall

ϕ = angle of internal friction of retained soil

X = SQUARE ROOT ($\text{COS}^2 B - \text{COS}^2 \phi$)

Figure 14.1 shows the earth pressure acting on a wall with sloped surcharge. The vertical effect of surcharge acting above the footing should not be included when considering overturning. The unit weight of soil (W) is supplied in the geotechnical report and should be used for design purposes. Unfactored dead loads and live loads are used to determine factor of safety against sliding and rotation.

Concrete design is based on the Strength Design Method (Load Factor Design) using AASHTO load factors, 1.3 for vertical earth pressure, 1.69 for lateral earth pressure, and 2.17 for lateral earth pressure from live load surcharge. Concrete strength shall be 3500 psi and reinforcing steel yield stress shall be 60,000 psi.

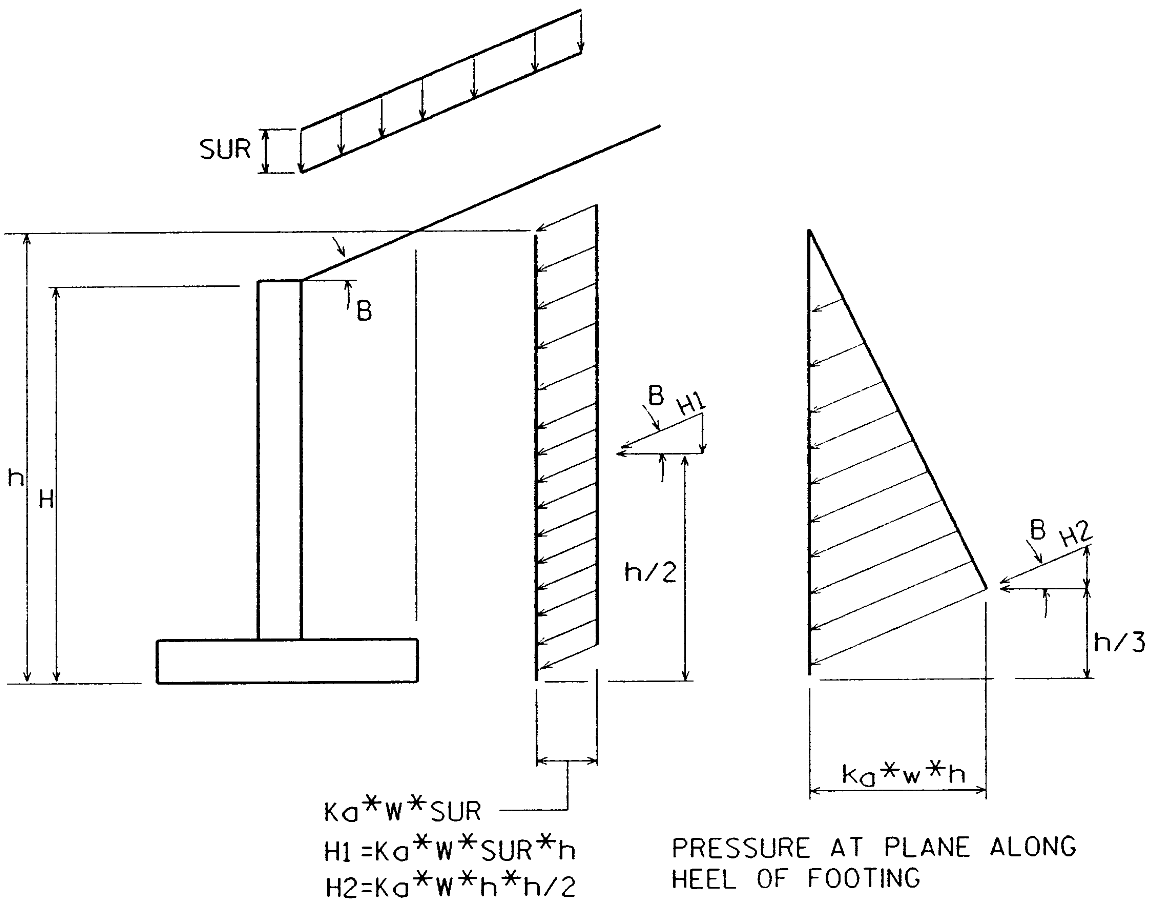


FIGURE 14.1

A. Overturning

| The minimum overturning Factor of Safety required is 1.5 for footings on piles or rock and 2.0 for footings on soil. When calculating overturning, moments are taken about the toe of spread footings and about the centerline of the front line of piles for pile footings.

B. Sliding

| The minimum sliding Factor of Safety required is 1.5 for spread footings on soil or rock and 1.0 for pile footings. The low safety factor of pile footings is used because the lateral resistance of piles given in Chapter 11.0 are based on lateral deflections which are less than what can be tolerated by retaining walls. Lateral resistance of piles increase rapidly with increased deflection.

Factors resisting sliding for Spread Footings on soil include the following. See Figure 14.2 for an example.

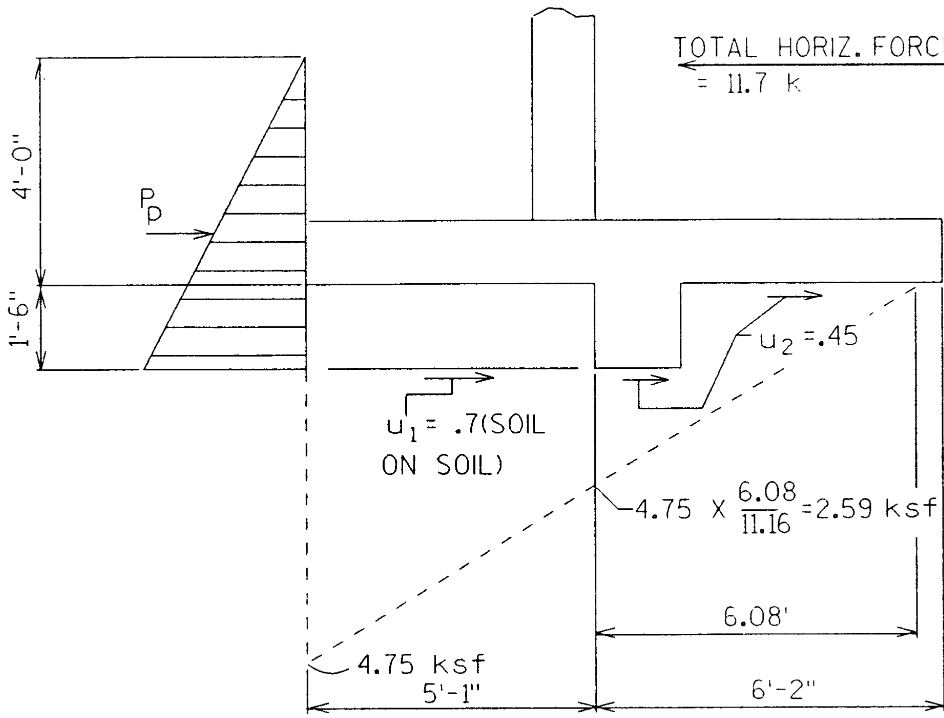
1. Passive earth pressure P_p in front of wall. For sand use 2.5 times weight of soil. For clay use .5 times unconfined compressive strength. The passive pressure of sand is taken as an equivalent fluid pressure. For clay it is taken as a uniform force over the depth.

2. Friction between soil and concrete. (μ_2)

   For Clay: Resisting Force = (Unconfined compressive strength) x .5
   For Sand: Resisting Force = W x Tan S
   Where: W = Vertical Load
   Tan S is coefficient of sliding friction. (Tan S = .45 is a safe value)

3. Friction between soil and soil (μ_1) in front of shear key is used. Multiply vertical load times the friction factor (Use .70 for sand).

4. Shear key. Use passive pressure of soil in front of shear key.



μ_1 = Coefficient of Friction for Soil on Soil (Tan ϕ = 0.7; for ϕ = 35°)

μ_2 = Coefficient of Friction for Concrete on Soil (TanS = 0.45)

Total Frictional Force $F = \mu_1 R_1 + \mu_2 R_2$

$= 0.7(1/2)(4.75 + 2.59)5.08 + 0.45 \times 1/2 \times 2.59 \times 6.08$

$= 13.03 + 3.54 = \underline{\underline{16.57^k}}$

Passive Pressure in Front of Wall = P_p

$P_P = 1/2\ Kp\ Wh^2$; $W = 0.120 k/ft^3$

$P_P = 1/2\ (2.5).12(5.5)^2 = \underline{\underline{4.54^k}}$

$$F.S. = \frac{16.57 + 4.54}{11.7} = \underline{\underline{1.80}} \quad O.K. > 1.5$$

FIGURE 14.2 - FACTOR OF SAFETY AGAINST SLIDING FOR SPREAD FOOTING

Factors resisting sliding for Pile Footings include the following:

1. Passive earth pressure in front of wall. Same as spread footing.

2. Horizontal component of battered piles. Maximum batter is 3 inches per foot.

3. Lateral resistance of battered or vertical piles in addition to horizontal component of battered piles. Refer to Chapter 11.0 for allowable lateral load capacity.

4. Do not use soil friction under the footing as consolidation of the soil may eliminate contact between the soil and footing.

For Spread Footings on rock, key the footing into the rock.

C. Stem Design

Use the following criteria when designing the stem.

1. Provide a constant batter of 2 inches minimum to the front face of wall. Use a seven inch minimum for walls over 30 feet. This is for appearance and to compensate for stem deflection and footing rotation. For stem heights from 16 feet to 26 feet inclusive the back face is battered 1/2 inch per foot. For stem heights of 28 feet and greater batter back face 3/4 inch per foot. The designer has the option to vary these values or to batter the front face only depending on site requirements.

2. The minimum stem thickness at the top is 12 inches. Stem thickness at the bottom is based on load requirements and/or batter.

3. No vertical stirrups are used in the stem.

4. Stem height is determined by site conditions. Refer to Figure 14.6.

5. Locate the stem on the footing to meet stability and soil pressure requirements. For walls on rock stability is the only factor that controls. Locate the stem to produce the most economical footing.

6. Check shear stress in wall at the base of the stem.

7. Use No. 4 bars at 1'-6" in front face of stem as longitudinal and vertical reinforcing for temperature reinforcement.

8. Design for moment at the base of the stem and where required for bar cut offs. Use one length of bar for walls of 10 feet height and under.

Loads from railings or parapets on top of the wall need not be applied simultaneously with loads from earth pressure. These are dynamic loads which are resisted by the mass of the wall and passive earth pressure.

D. Footing Design

Use the following criteria when designing the footing.

1. Minimum footing thicknesses are:
   1'-6 - Spread footings
   2'-0 - Spread footing with stem height plus footing thickness greater than 10 feet.
   2'-0 - Pile footings

2. Place the bottom of footings 4 feet below the finished ground line. If the finished ground line is on a grade, the bottom of footings may be sloped to a maximum grade of 20 percent. If the grade exceeds 20 percent, place the footings level and use steps.

3. Maximum pile spacing in any row is 8 feet for timber and 10 feet for C.I.P.C. and steel H-piles.

4. Maximum pile batter is 3 in 12.

5. Embed piles 6 inches into footing. Place bar steel on top of piles.

6. For spread footings place bar steel with 3 inches clear from bottom of footing. Use 2 inches clear for edge distance.

7. Design for footing moment at the face of the stem based on vertical loads and resultant soil pressure. No bar steel is provided if the required area per foot is less than .05 square inches. This practice has been done for years with no problems.

8. A design for heel moment without considering the upward soil or pile reaction is not required unless such a condition actually exists.

9. For toe, design for shear at a distance from the face of the stem equal to the effective "d" distance of the footing. For heel, design for shear at face of stem.

E. Shear Key Design

Use the following criteria when designing the shear key:

1. Place shear key in line with stem except under severe loading conditions.

2. The key width is 1'-0" in most cases. The minimum key depth is 1'-0".

3. Place shear key in unformed excavation against undisturbed material.

4. Shear keys are analyzed for the forces shown in Figure 14.2.

5. The shape of shear keys in rock is determined by the site conditions.

F. Design Steps

1. Determine the tentative size of the wall.

2. Determine the magnitude of all forces acting on the wall.

3. Determine the stability of the wall against sliding and overturning.

4. Determine maximum foundation pressure.

5. Reproportion wall if necessary and begin at Step 2 again.

6. Design the bar steel for stem, toe and heel.

7. Reproportion wall if necessary and begin at Step 2 again.

G. Miscellaneous Design Information

1. If a wall is adjacent to a traveled roadway or sidewalk, use pipe drains in back of the wall instead of weep holes. Use a six-inch pipe underdrain with the flow line at the bottom of a two foot square course of fine aggregate. Discharge this system into a storm sewer system or ditch. For rehab of existing retaining walls, provide plan details to replace inadequate pipe underdrain systems. Use a minimum slope of .005 feet/foot for pipe underdrains.

2. Optional transverse construction joints are permitted in the footing with a minimum spacing of three panel lengths. Footing joints should be offset a minimum of 1'-0 from wall joints. Run bar steel thru footing joints.

3. Granular backfill is placed behind the wall only to a vertical plane 18 inches beyond the face of footing. Lower limit is to the bottom of the footing.

(2) Design Tables for Cast-in-Place Concrete Cantilever Walls

The following design tables are based on WisDOT design criteria and the following assumptions:

Angle of Internal Friction of Retained Earth - 34 degrees
Allowable soil pressure - 4 ksf
Allowable rock pressure - 16 ksf
Allowable pile load - 30 tons/pile (12" timber)
Concrete strength - 3.5 k.s.i.
Bar steel yield stress - 60.0 k.s.i.

The tables are for preliminary design information only and should be verified using properties of soils at the site prior to being used for detailed designs. They should not be used if the Angle of "Internal Friction of Retained Earth" is different than 34 degrees. They assume the unit weight of soil is 120 pounds per cubic foot. Active earth pressure is by Coulomb's equation for the resultant parallel to the backfill slope. No hydraulic pressure is assumed. Two feet of fill above top of footing was assumed in creating Tables 14.1-14.4.

h = (Wall Height)

		0' SURCHARGE		2' SURCHARGE		2:1 BACKSLOPE	
h	T = 1'-0' BATTER in./ft.	SIZE	SPACE	SIZE	SPACE	SIZE	SPACE
2	0	4	12	4	12	4	12
4	0	4	12	4	12	4	12
6	0	4	12	4	12	4	12
8	0	4	12	5	12	5	12
10	0	5	12	6	11	6	12
12	0	6	10	8	12	8	12
14	0	8	11	9	9	9	9
16	.5	7	11	8	10	8	10
18	.5	8	11	9	10	10	12
20	.5	9	10	10	10	11	11
22	.5	10	10	11	9	11	8
24	.5	11	10	11	8	(1) 11	9
26	.5	11	8	(1) 11	9	(1) 11	8
28	.75	11	9	11	7	(2) 11	9

Refer to Figure 14.6 for details. The minimum requirement of bar steel area is based on AASHTO 8.17.2001

(1) Batter is .75 in./ft.
(2) Batter is 1.00 in./ft.

TABLE 14.0 STEM REINFORCEMENT FOR RETAINING WALLS

0' SURCHARGE - NO PILES - ON SOIL

H (ft)	B	A	D	T=1'-0 BATTER (in/ft)	TOE STEEL			HEEL STEEL			MAX. P (ksf)	SHEAR KEY
					SIZE	SPACING	LENGTH	SIZE	SPACING	LENGTH		
4	1-9	6	1-6	0	--	--	--	--	--	--	1.2	NO
6	2-6	8	1-6	0	--	--	--	--	--	--	1.8	NO
8	3-4	10	1-6	0	--	--	--	--	--	--	2.3	NO
10	4-2	1-0	1-6	0	--	--	--	4	12	3-2	2.8	NO
12	5-0	1-2	2-0	0	4	12	1-11	4	12	3-10	3.3	NO
14	5-11	1-4	2-0	0	4	12	2-1	5	12	4-10	3.7	NO
16	6-11	1-8	2-0	.5	4	12	2-5	5	11	5-5	4.0	NO
18	8-0	2-3	2-0	.5	5	12	3-0	6	11	5-8	4.0	NO
20	9-0	3-0	2-0	.5	6	12	4-1	7	12	6-5	4.0	NO
22	10-0	4-0	2-0	.5	7	12	5-5	7	12	6-4	4.0	YES
24	11-0	4-9	2-0	.5	8	11	6-8	7	12	6-6	4.0	YES
26	12-3	5-9	2-3	.5	9	11	8-3	7	11	6-8	4.0	YES
28	13-6	6-9	2-6	.75	10	11	10-0	7	12	6-11	4.0	YES
30	15-0	7-3	2-6	.75	11	12	11-3	7	10	7-3	4.0	YES

T = Dimension at top of stem.

2' SURCHARGE - NO PILES - ON SOIL

H (ft)	B	A	D	T=1'-0 BATTER (in/ft)	TOE STEEL			HEEL STEEL			MAX. P (ksf)	SHEAR KEY
					SIZE	SPACING	LENGTH	SIZE	SPACING	LENGTH		
4	2-3	6	1-6	0	--	--	--	--	--	--	1.6	NO
6	3-2	8	1-6	0	--	--	--	--	--	--	2.1	NO
8	4-0	10	1-6	0	--	--	--	4	12	3-2	2.7	NO
10	4-11	1-0	1-6	0	4	12	1-9	4	10	3-11	3.1	NO
12	5-10	1-3	2-0	0	4	12	2-0	5	12	4-10	3.6	NO
14	6-9	1-6	2-0	0	4	12	2-3	6	12	5-10	3.9	NO
16	7-10	2-0	2-0	.5	4	11	2-9	6	10	5-10	4.0	NO
18	8-11	2-9	2-0	.5	6	12	3-9	7	12	6-8	4.0	NO
20	10-0	3-6	2-0	.5	7	12	4-11	7	12	6-11	4.0	YES
22	11-2	4-3	2-0	.5	7	10	5-8	7	10	7-4	4.0	YES
24	12-4	5-0	2-0	.5	9	12	7-6	8	11	8-3	4.0	YES
26	13-9	5-9	2-3	.75	9	10	8-3	8	12	9-2	4.0	YES
28	14-11	7-0	2-6	.75	10	10	10-3	8	12	8-2	4.0	YES
30	16-6	7-6	2-9	.75	10	10	10-9	8	11	9-2	4.0	YES

Max. P = Maximum soil pressure under footing, Kips per Sq. Ft.
Refer to Figure 14.4 for definitions. Use minimum shear key where required.

TABLE 14.1

RETAINING WALLS - ON SOIL - LEVEL BACKFILL

0' SURCHARGE - PILES

H (ft)	B	A	D	T=1'-0 BATTER (in/ft)	TOE STEEL			HEEL STEEL			PS1	PILE SPACING			PS2
					SIZE	SPACE	LENGTH	SIZE	SPACE	LENGTH		1	2	3	
6	3-8	8	2-0	0	--	--	--	4	12	3-0	1-0	8-0	--	8-0	--
8	4-5	10	2-0	0	--	--	--	4	12	3-7	1-6	8-0	--	8-0	--
10	5-0	1-0	2-0	0	--	--	--	4	12	4-0	2-0	8-0	--	8-0	--
12	5-9	1-6	2-0	0	4	12	2-3	4	10	4-3	2-6	8-0	--	8-0	--
14	6-6	2-0	2-0	0	4	12	2-9	5	11	4-9	3-0	8-0	--	8-0	--
16	7-3	2-6	2-0	.5	6	12	3-6	5	10	4-5	3-6	7-9	--	8-0	--
18	8-6	3-0	2-0	.5	7	11	4-5	7	12	6-0	4-0	6-6	--	8-0	--
20	9-3	3-3	2-3	.5	7	10	4-8	7	10	6-5	4-6	5-5	--	8-0	--
22	10-0	3-6	2-3	.5	7	10	4-11	8	10	7-6	2-6	5-6	8-0	8-0	2-6
24	10-9	4-3	2-3	.5	8	10	6-2	8	10	7-5	2-6	5-0	6-4	6-4	2-9
26	11-6	5-2	2-3	.5	9	11	7-8	8	10	7-2	2-9	4-6	4-0	4-0	3-0
28	12-6	5-6	2-6	.75	9	10	8-0	9	12	8-7	3-0	3-0	3-0	3-0	3-6

NOTE: For walls over 28' with 0' surcharge use 4 rows of piles and make individual design of footing.
T = Dimension at top of stem.

2' SURCHARGE - PILES

H (ft)	B	A	D	T=1'-0 BATTER (in/ft)	TOE STEEL			HEEL STEEL			PS1	PILE SPACING			PS2
					SIZE	SPACE	LENGTH	SIZE	SPACE	LENGTH		1	2	3	
6	4-2	9	2-0	0	--	--	--	4	12	3-5	1-6	8-0	--	8-0	--
8	4-10	1-0	2-0	0	--	--	--	4	12	3-10	2-0	8-0	--	8-0	--
10	5-7	1-3	2-0	0	--	--	--	5	12	4-7	2-6	8-0	--	8-0	--
12	6-4	1-6	2-0	0	4	12	2-3	5	10	5-1	3-0	8-0	--	8-0	--
14	7-1	1-9	2-0	0	4	12	2-6	7	12	6-6	3-6	7-2	--	8-0	--
16	7-10	2-3	2-0	.5	6	12	3-3	7	12	5-7	4-0	6-1	--	8-0	--
18	9-6	2-9	2-3	.5	6	10	3-9	8	12	8-2	4-6	5-3	--	8-0	--
20	9-8	3-3	2-3	.5	7	10	4-8	8	10	7-6	2-6	5-6	8-0	8-0	2-6
22	11-0	4-6	2-3	.5	8	10	6-5	9	12	8-7	2-6	4-9	5-9	5-9	2-6
24	11-9	4-9	2-6	.5	9	12	7-3	9	11	8-10	2-9	4-6	4-0	4-0	2-9
26	12-3	5-3	2-6	.75	9	11	7-9	9	12	8-8	3-0	3-0	3-0	3-0	3-0

Refer to Figure 14.5 for definitions

TABLE 14.2

RETAINING WALLS - ON PILES - LEVEL BACKFILL

0' SURCHARGE ON ROCK

H (ft)	B	A	D	T=1'-0 BATTER (in/ft)	TOE STEEL SIZE	TOE STEEL SPACING	TOE STEEL LENGTH	HEEL STEEL SIZE	HEEL STEEL SPACING	HEEL STEEL LENGTH	MAX. P (ksf)
4	1-9	6	1-6	0	--	--	--	--	--	--	1.2
6	2-6	8	1-6	0	--	--	--	--	--	--	1.8
8	3-4	10	1-6	0	--	--	--	4	12	2-6	2.3
10	4-2	1-0	1-6	0	4	12	1-9	4	12	3-2	2.8
12	5-0	1-3	2-0	0	4	12	2-0	4	12	3-9	3.3
14	5-10	1-6	2-0	0	4	12	2-3	5	12	4-7	3.7
16	6-8	1-9	2-0	.5	4	12	2-6	5	12	4-7	4.1
18	7-6	2-0	2-0	.5	5	12	2-9	6	11	5-5	4.6
20	8-4	2-6	2-0	.5	6	12	3-6	7	12	6-3	4.8
22	9-2	3-0	2-0	.5	7	12	4-5	7	12	6-6	5.1
24	10-0	3-6	2-0	.5	7	11	4-11	7	10	6-9	5.4
26	10-10	4-0	2-0	.5	8	10	5-11	8	11	7-8	5.7
28	12-0	4-4	2-3	.75	9	12	6-10	9	12	9-3	5.8
30	13-0	4-9	2-3	.75	9	10	7-3	9	11	9-2	6.0

2' SURCHARGE ON ROCK

H (ft)	B	A	D	T=1'-0 BATTER (in/ft)	TOE STEEL SIZE	TOE STEEL SPACING	TOE STEEL LENGTH	HEEL STEEL SIZE	HEEL STEEL SPACING	HEEL STEEL LENGTH	MAX. P (ksf)
4	2-3	6	1-6	0	--	--	--	--	--	--	1.6
6	3-2	9	1-6	0	--	--	--	--	--	--	2.0
8	4-0	1-0	1-6	0	4	12	1-9	4	12	3-0	2.6
10	4-10	1-4	1-6	0	4	12	2-1	4	12	3-6	3.0
12	5-8	1-8	2-0	0	4	12	2-5	4	11	4-0	3.5
14	6-6	2-0	2-0	0	4	12	2-9	5	11	4-9	3.9
16	7-4	2-3	2-0	.5	5	12	3-0	6	12	5-1	4.4
18	8-2	2-6	2-0	.5	6	12	3-6	7	12	6-2	4.8
20	9-0	2-11	2-0	.5	7	12	4-4	7	12	6-6	5.2
22	9-10	3-3	2-0	.5	7	12	4-8	8	12	7-7	5.6
24	10-9	3-8	2-0	.5	8	12	5-7	9	12	8-10	5.8
26	11-7	4-0	2-3	.75	9	11	5-11	9	12	9-3	6.2
28	12-5	4-4	2-3	.75	9	11	6-10	9	10	9-2	6.6
30	13-4	4-9	2-6	.75	9	10	7-3	9	10	9-7	6.9

TABLE 14.3

RETAINING WALLS - ON ROCK - LEVEL BACKFILL

2:1 BACKSLOPE - ON SOIL

H (ft)	B	A	D	T=1'-0 BTR (in/ft)	TOE STEEL SIZE	TOE STEEL SPACE	TOE STEEL LENGTH	HEEL STEEL SIZE	HEEL STEEL SPACE	HEEL STEEL LENGTH	MAX. P (ksf)	SHEAR KEY	DSK	XK
4	2-3	9	1-6	0	--	--	--	--	--	--	1.3	NO	--	--
6	3-3	1-6	1-6	0	4	12	2-3	--	--	--	1.7	NO	--	--
8	4-6	2-0	1-6	0	4	12	2-9	4	12	2-6	2.0	NO	--	--
10	5-9	2-6	1-6	0	4	10	3-3	4	12	3-3	2.4	YES	1-0	2-6
12	7-0	3-0	2-0	0	5	12	3-9	4	10	4-0	2.9	YES	1-0	3-0
14	8-3	3-6	2-0	0	6	11	4-6	6	12	5-4	3.3	YES	1-0	3-6
16	9-6	4-3	2-0	.5	7	12	5-8	6	12	5-3	3.5	YES	1-0	4-3
18	11-0	5-0	2-0	.5	8	11	6-11	7	12	6-6	3.7	YES	1-6	5-0
20	12-9	5-9	2-0	.5	9	10	8-3	8	12	8-1	3.7	YES	1-6	5-9
22	14-3	6-9	2-3	.5	10	12	10-6	9	12	9-4	3.9	YES	2-0	10-0
24	16-9	7-0	2-6	.75	10	10	10-3	10	11	12-6	4.0	YES	2-9	12-0
26	18-0	9-0	3-0	.75	11	10	13-0	9	12	10-3	4.0	YES	2-9	12-0
28	20-0	10-0	3-3	1.0	11	9	14-0	9	12	11-2	4.0	YES	2-9	14-0
30	22-0	11-3	3-9	1.0	11	8	15-3	9	12	11-3	4.0	YES	2-9	16-0

T = Dimension at top of stem.

2:1 BACKSLOPE - ON PILES

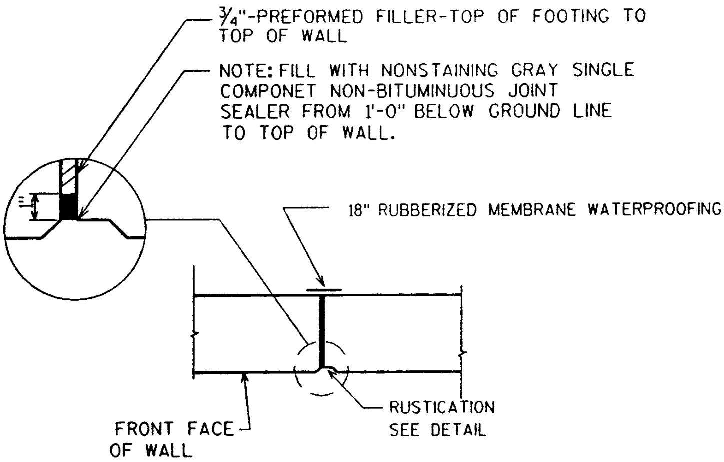
H (ft)	B (ft)	A (in)	D (ft)	T=1'-0 BATTER (in/ft)	TOE STEEL SIZE	TOE STEEL SPACE	TOE STEEL LENGTH	HEEL STEEL SIZE	HEEL STEEL SPACE	HEEL STEEL LENGTH	PS1	PILE SPACING 1	PILE SPACING 2	PILE SPACING 3	PS2
6	4-6	2-0	2-0	0	4	12	2-9	--	--	--	2-0	8-0	--	8-0	--
8	5-7	2-6	2-0	0	4	12	3-3	4	12	3-1	2-6	8-0	--	8-0	--
10	6-8	3-0	2-0	0	5	12	3-9	4	12	3-8	3-0	8-0	--	8-0	--
12	7-9	3-6	2-0	0	7	12	4-11	6	12	4-10	3-6	8-0	--	8-0	--
14	8-10	4-3	2-0	0	8	11	6-2	6	11	5-2	4-6	7-0	--	7-0	--
16	9-11	4-9	2-0	.5	8	11	6-8	7	11	5-9	2-6	7-0	7-0	7-0	2-6
18	11-0	5-3	2-0	.5	9	12	7-9	8	11	6-11	2-6	4-0	4-0	4-0	2-6
20	12-3	6-0	2-3	.5	9	10	8-6	8	10	7-4	3-0	2-6	2-6	2-6	3-0

NOTE: For walls over 20' use 4 rows of piles and make individual design of footing.

Refer to Figures 14.4 or 14.5 for definitions.

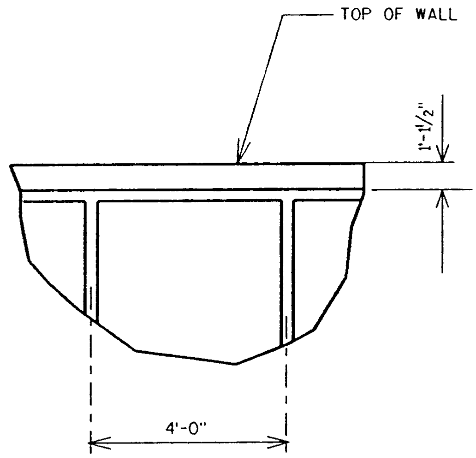
TABLE 14.4

RETAINING WALL - 2:1 SLOPING BACKFILL

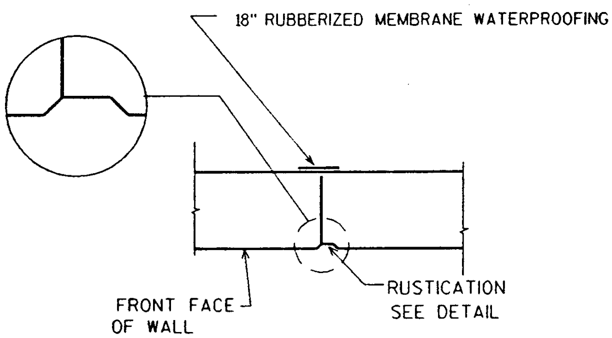


VERTICAL EXPANSION JOINT

DO NOT RUN ANY BAR STEEL THRU JOINT
MAX. SPA. OF JOINT 90'

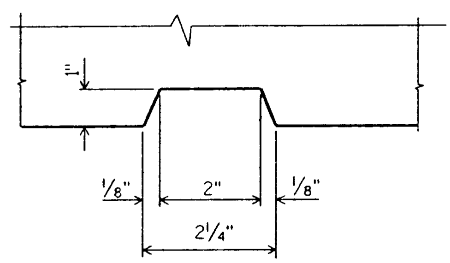


ELEVATION OF WALL



VERTICAL CONTRACTION JOINT

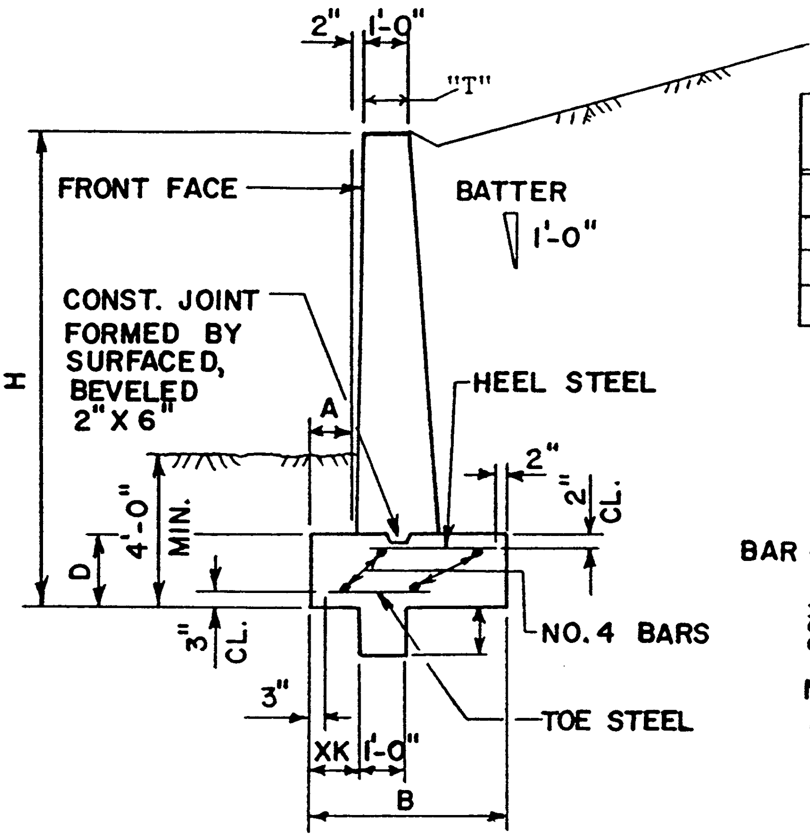
DO NOT RUN ANY BAR STEEL THRU JOINT
MAX. SPA. OF JOINT = 30'-0"



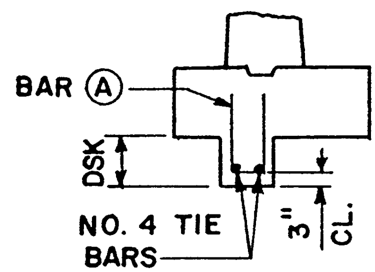
RUSTICATION DETAIL

TYPICAL HORIZ. AND VERTICAL

FIGURE 14.3 RETAINING WALL DETAILS



DSK	BAR (A) SIZE	SPACING
1-0	4	2-0
1-6	4	1-0
2-0	4	9
2-9	6	10



SHEAR KEY REINFORCEMENT

FIGURE 14.4

RETAINING WALL - NO PILES

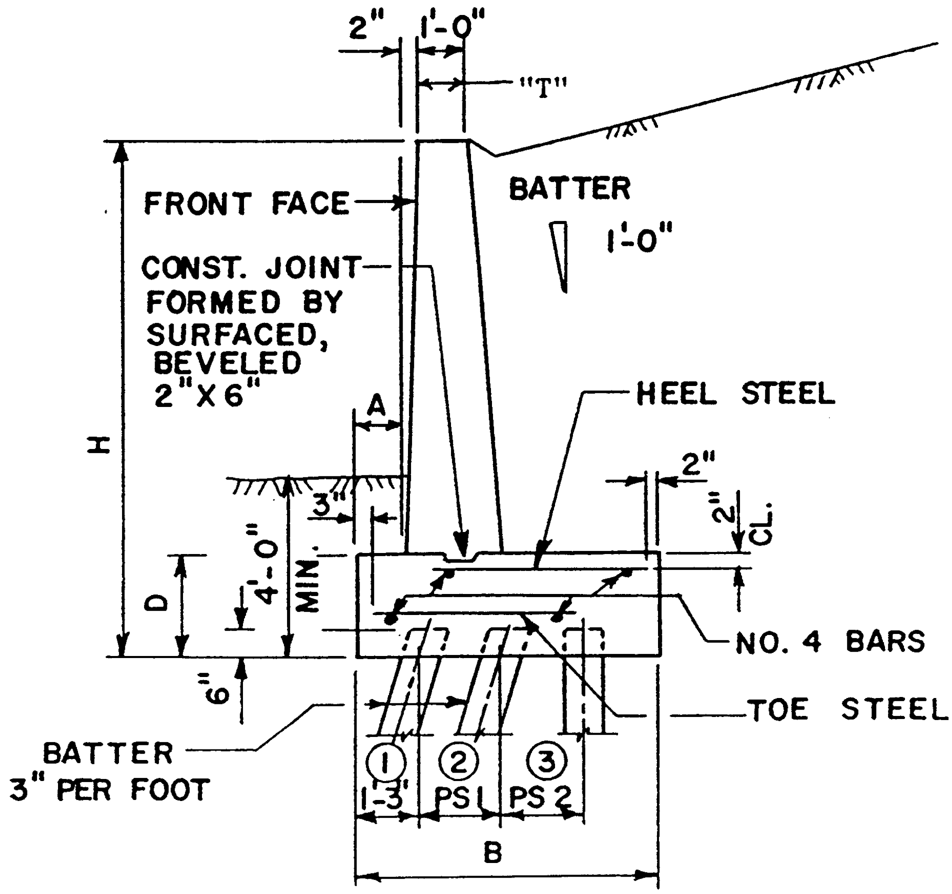
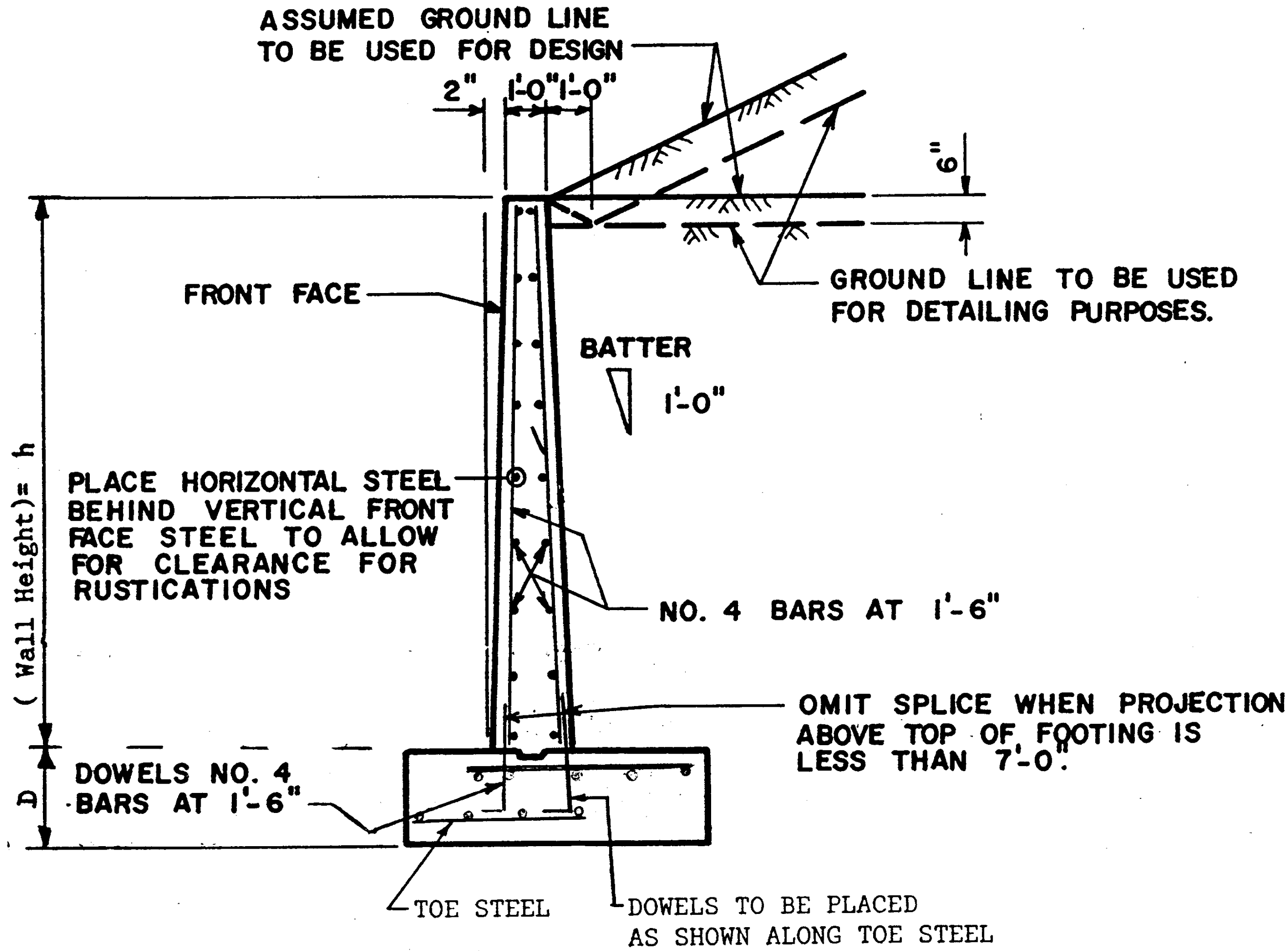


FIGURE 14.5

RETAINING WALL - PILES



STEM REINFORCEMENT FOR RETAINING WALLS

FIGURE 14.6

(3) Summary of Design Safety Factors and Requirements

Requirements

a. Safety Factors

Overturning

Spread footing	=	2.0
Pile footing	=	1.5

Sliding

Spread footing	=	1.5
Pile footing	=	1.0
Global	=	1.3

b. Foundation Design Parameters

Use values provided by WisDOT

c. Concrete Design Data

f'c 3500. psi

fy = 60,000 psi

Load factors

Vertical earth pressure	1.3
Lateral earth pressure	1.69
Live load surcharge	2.17

d. Traffic Surcharge

Traffic live load surcharge = 2 feet = 240 lb/ft^2

e. Retained Soil

Unit weight = 120 lb/ft^3

Angle of internal friction

Use value provided by WisDOT.

f. Soil Pressure Theory

Rankine's Theory or Coulombs Theory at the discretion of the designer.

14.3(B) POST AND PANEL WALLS

Post and panel walls are comprised of vertical elements (usually steel H piles) and concrete panels which extend between the vertical elements. The panels are usually constructed of precast reinforced concrete although precast prestressed concrete is also a possibility. Precast prestressed concrete can also be used for the vertical elements.

Post and panel walls should be considered if minimum environmental damage and/or disturbances to the site from construction procedures is critical. Post and panel walls may also be used when an irregular rock surface and/or rock near the surface exists at the wall location since the holes for the posts can be drilled into the rock.

(1) Design Procedure for Post and Panel Walls

AASHTO "Standard Specifications for Highway Bridges" Article 5.6 covers the design of post and panel walls. A simplified earth pressure distribution diagram is shown in AASHTO 5.6.2 for permanent post and panel walls. Another method that may be used is the "Conventional Method" as described in "United States Steel Sheet Piling Design Manual", February, 1974. This method must be modified for the fact that it is based on continuous vertical wall elements whereas, post and panel walls have discrete vertical wall elements. Using "Broms" method for designing drilled shafts is also acceptable. WisDOT provides the soil design parameters including cohesive values, angles of internal friction, angles of wall friction, soil densities, and water table elevations.

The maximum spacing between vertical supporting elements (posts) depends on the wall height and the design parameters of the foundation soil. Spacing of 6 to 12 feet is typical. The posts (vertical elements) are set in drilled holes and concrete is placed in the hole after the post is set. The post system must be designed to handle maximum bending moment along length of embedded shaft. The maximum bending moment at any level in the facing can be determined from formulas in AASHTO 5.6.6. The minimum panel thickness allowed is 6 inches.

The diameter of the drilled shaft is also dependent on the wall height and the design parameters of the foundation soil. The larger the diameter of the drilled shaft the smaller will be the required embedment of the shaft. The designer should try various shaft diameters to optimize the cost of the drilled shaft considering both material cost and drilling costs. Note that drilling costs are a function of both hole diameter and depth.

If the vertical elements are steel they shall be shop painted. Wall panels are usually given a special surface treatment created by brooming or tining vertically, using form liners, or using a pattern of rustication strips. The portion of the panel receiving the special treatment may be recessed, forming a border around the treated area. Concrete paints or stains may be used for color enhancements.

When panel heights exceed 15 feet anchored walls may be needed. Anchored wall design must be in accordance with AASHTO Article 5.7. Anchors for permanent walls shall be fully encapsulated over their entire length. The anchor hardware shall be designed to have a corrosion resistance durability to ensure a minimum design life of 75 years.

The concrete for post and panel walls shall have a 28 day compressive strength of 4000 psi if non-prestressed and 5000 psi if prestressed except for the drilled shafts. Concrete for the drilled shafts shall have a 28 day compressive strength of 3500 psi. Reinforcement shall be uncoated Grade 60 in drilled shafts. In lieu of drainage aggregate a membrane may be used to seal the joints between the vertical elements and concrete panels to prevent water leakage. The front face of post and panel walls shall be battered 1/4" per foot to account for short and long term deflection.

(2) Summary of Design Safety Factors and Requirements

Requirements

a. Safety Factors

Global = 1.3
Passive Pressure Reduction Kp' = 2/3*Kp

b. Foundation Design Parameters
Use values provided by WisDOT

c. Concrete Design Data

f'c = 3500 psi (for drilled shafts)
f'c = 4000 psi (non-prestressed panel)
f'c = 5000 psi (prestressed panel)
fy = 60,000 psi
Load Factors

Vertical earth pressure	1.3
Lateral earth pressure	1.69
Live load surcharge	2.17

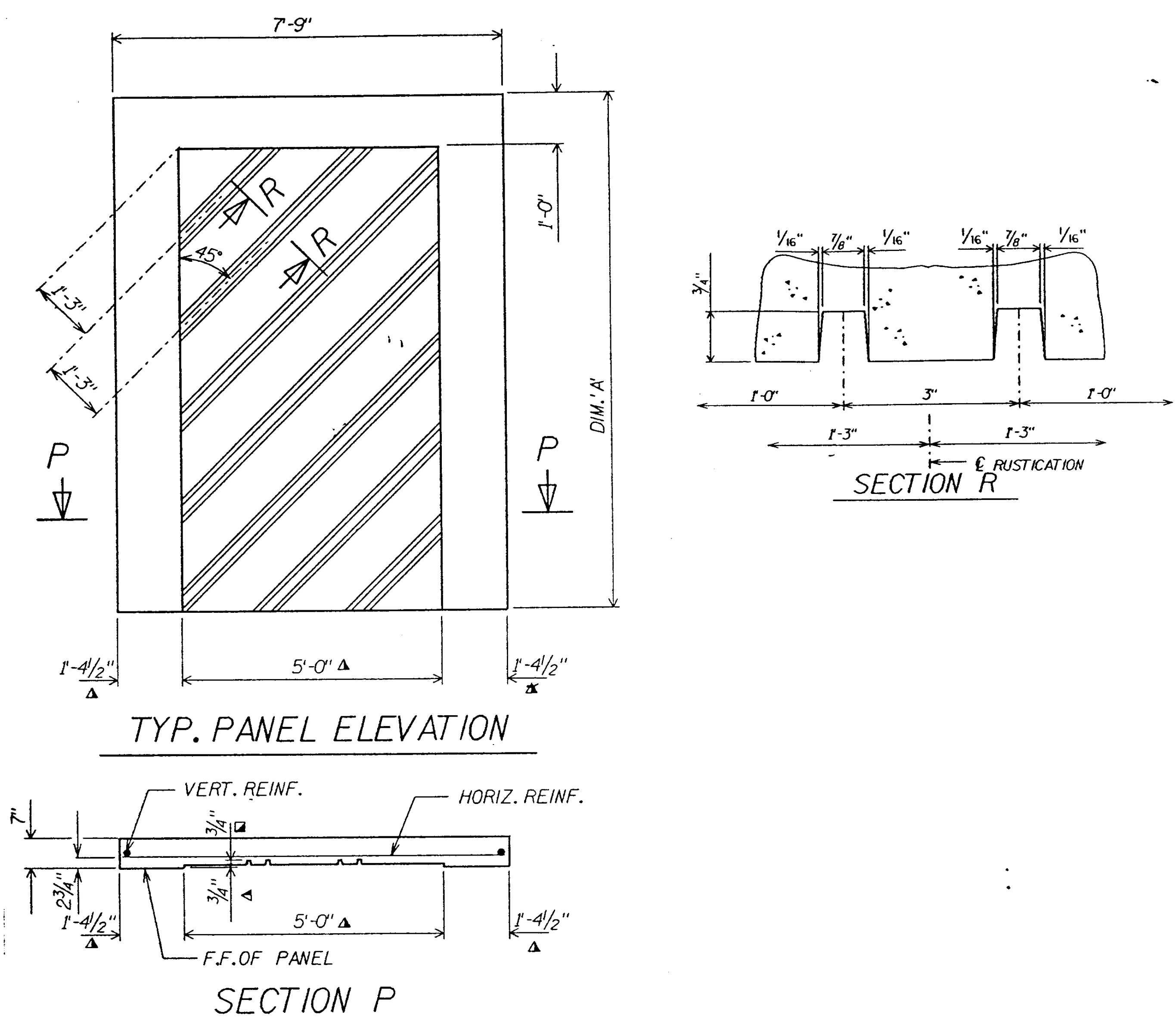
d. Traffic Surcharge
Traffic live load surcharge = 2 feet = 240 lb/ft^2

e. Retained Soil
Unit weight = 120 lb/ft^3
Angle of internal friction
Use value provided by WisDOT.

f. Soil Pressure Theory
Rankine's Theory or Coulombs Theory at the discretion of the designer.

g. Design Life for Anchorage Hardware
75 year minimum

h. Steel Design Properties (H-piles)
Minimum yield strength = 36,000 psi



▲ FRONT FACE OF PANEL SHALL HAVE A ¾" DEEP RECESS AS SHOWN.
RECESS SHALL BE BROOMED OR TINED VERTICALLY.

◪ RUSTICATIONS (@ 45°) SHOWN IN RECESS ARE OPTIONAL TO BROOMED OR TINED FINISH.

FIGURE 14.7
EXAMPLE OF PANEL FOR POST AND PANEL WALL SYSTEM

14.3(C) GABION WALLS

Gravity retaining walls may be constructed from rock filled wire baskets commonly called gabions or gabion baskets. The gabions are manufactured from a heavy wire mesh with a nominal 3 inch opening and are formed into rectangular baskets. Common basket sizes include a standard depth of 3 feet, heights of 1, 1 1/2 or 3 feet, and lengths of 6, 9, or 12 feet. Individual baskets are placed on the prepared earthen surface, reinforced with internal tie wires, and filled with a select stone ranging in size from 4 to 10 inches. After the baskets are filled, the lids are closed and wired shut to form a relatively rigid block. Succeeding rows of gabions are laced to the filled underlying gabions and are filled in the same manner until the wall reaches the design height. Proprietary gabion basket manufacturers will supply details for the wires, lacing, and lid closure. However, the manufacturers usually do not provide internal or external wall design. External stability considerations are usually determined by WisDOT.

Gabion walls can be used for a variety of applications. Walls on grades may be accommodated by either putting steps in the wall or by sloping the base of the wall. Gabion walls on grades of 5% or more have a more pleasing appearance if steps are utilized. Gabion walls may be constructed adjacent to streams or lakes so that at least a portion of the wall may be below water line. For this application it is normally necessary to dewater the wall site during construction. For water installations, all gabion walls should be protected against erosion or scour by the use of riprap or other suitable protection. Gabion walls may also be constructed along a curved alignment. However, sharp curves with a radius of less than 25 feet may be difficult to construct and should be avoided. A layer of Type DF geotextile fabric should be placed on the back side of all gabion walls prior to backfilling to prevent soil migration and loss. The minimum embedment for Gabion walls is 1'-6.

The durability of a gabion wall is dependent upon maintaining the integrity of the gabion baskets. Galvanized steel wire is required for all gabion installations. In areas of high corrosion potential due to soil, water, salt spray, or abrasion conditions, a polyvinyl chloride coating should be required in addition to galvanizing. Conditions at individual sites should be assessed to determine corrosion potential. Although gabions are manufactured from a heavy gage wire, there is a potential for damage due to vandalism. While no known case of such vandalism has occurred on any existing WisDOT gabion walls, the potential for such action should be considered at specific sites.

In gabion wall design, the mass of a wall will increase disproportionately with increases in height. In other words, doubling the height of a wall will more than double the mass of the wall. The ratio of the base width to height will vary, but in no circumstances should this value fall below .5. In practice, this value will normally range from .5 to .75 depending on backslope, surcharge and angle of internal friction of retained soil. Gabion walls have shown good economy for low to moderate heights but lose this economy as height increases. A height of about 18 feet should be considered as a practical limit for gabion walls.

Gabion walls should be designed in cross section with a horizontal base and a set back of 4 to 6 inches at each basket layer. This setback is an aid to construction and presents a more pleasing appearance. The use of a tipped wall base should not be allowed except in special circumstances.

A geotechnical investigation and analysis is conducted by WisDOT to determine soil design parameters for retained and foundation soils. Consolidation potential due to wall loads is considered when determining foundation design parameters.

The appearance of gabion walls is often a topic of debate. Some people find the exposed wire of the baskets objectionable while other consider the appearance of the stone pleasing. Using special colored stones such as red granite or lava rock at the exposed faces will appreciably enhance the appearance. At close range, the wire baskets are obviously visible. As distance increases, the visibility of the wires decreases appreciably. The rough texture of the gabion baskets provide an attractive surface for climbing vines and plants. Plantings of this type at the base of the wall may provide a more natural appearance within a few seasons.

(1) Design Procedure for Gabion Walls

The design of gabion walls is not specifically covered by AASHTO "Standard Specifications for Highway Bridges" but they shall be designed in accordance with the applicable portions of AASHTO Article 5.9, "Prefabricated Modular Wall Design".

Design of a gabion wall must consider loads placed on that wall by the retained soil and any surcharges. Resistance to these loads are developed by proportionating the cross sectional area of the wall to achieve a sufficient mass to ensure stability. The analysis proceeds by computing resisting loads due to mass and dividing these by corresponding driving loads due to soil and surcharge pressures. Sliding and rotation should be considered for the full height wall and at each gabion layer in the wall. The minimum acceptable safety factors are 1.5 for sliding and 2.0 for rotation about the toe.

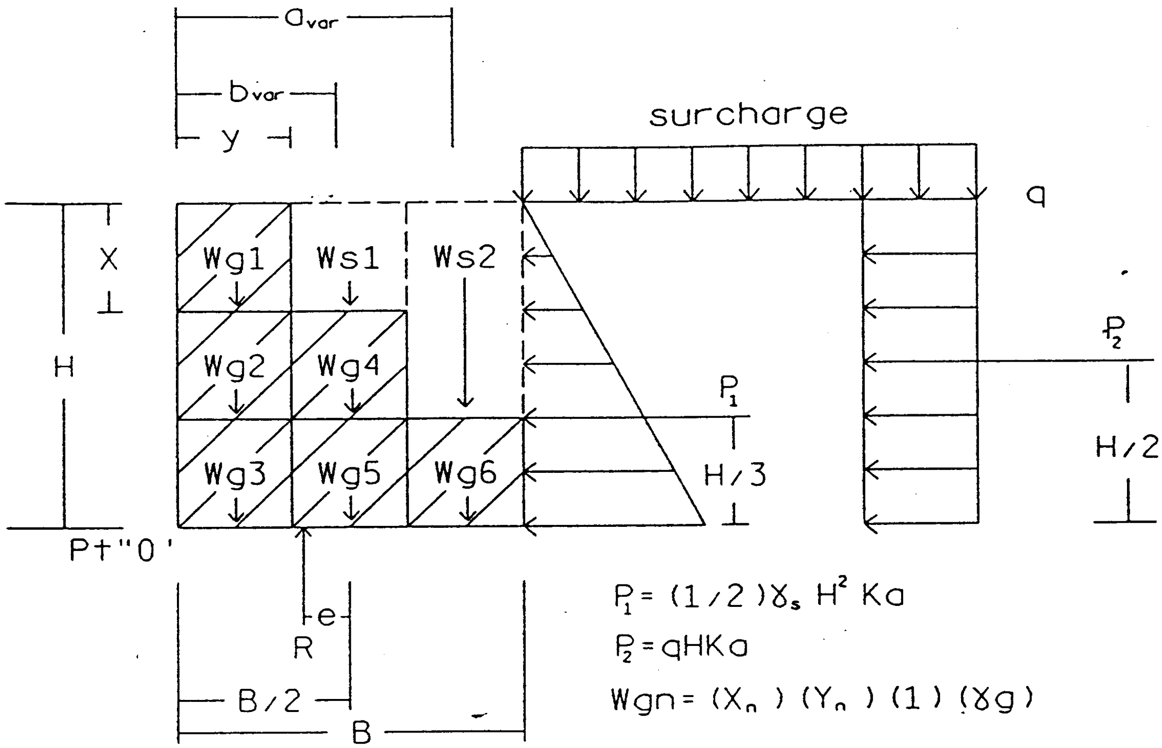
In additions, the base pressure of the wall cannot exceed the allowable bearing capacity of the foundation soil. Wall base pressure may be determined by using the Meyerhoff method in which vertical loads are distributed over a base area reduced for eccentricity. This method is shown on the accompanying drawings. More precise base pressures may be determined by a static analysis of all forces acting about location of the resultant. Global stability may be determined by conventional soil mechanic methods or programs. A safety factor of at least 1.3 should be attained for this condition.

Lateral earth pressures are determined by multiplying vertical loads by the coefficient of active earth pressure (Ka). This Ka value may be determined by either the Rankine method or the Coulomb method at the discretion of the designer.

In addition to the actual weight of the gabions, any earth backfill bearing directly on the gabions should be included as part of the wall system. Lateral earth pressure should be assumed to act on a vertical plane rising from the back of the wall base. These conditions are illustrated on the accompanying figures.

Gabion wall analysis is often simplified by separating the wall into individual sections based on gabion placement. Surcharge loads should be added when determining driving loads but should not be included when computing resisting values.

FIGURE 14.8
Horizontal Backslope with Surcharge



Safety Factor Against Sliding

$$S.F. = \frac{\text{Resisting Forces}}{\text{Driving Forces}} = \frac{[\Sigma\ Wgn + \Sigma\ Wsn]\ \text{Tan}\ \phi}{P_1 + P_2} = 1.5$$

Safety Factor Against Rotation about Pt. "0'"

$$S.F. = \frac{\text{Resisting Moments}}{\text{Driving Moments}} = \frac{M_R}{M_D} = \frac{(\Sigma\ Wgn)\ (b_n) + \Sigma Wsn\ (a_n)}{P_1(H/3) + P_2(H/2)} = 2.0$$

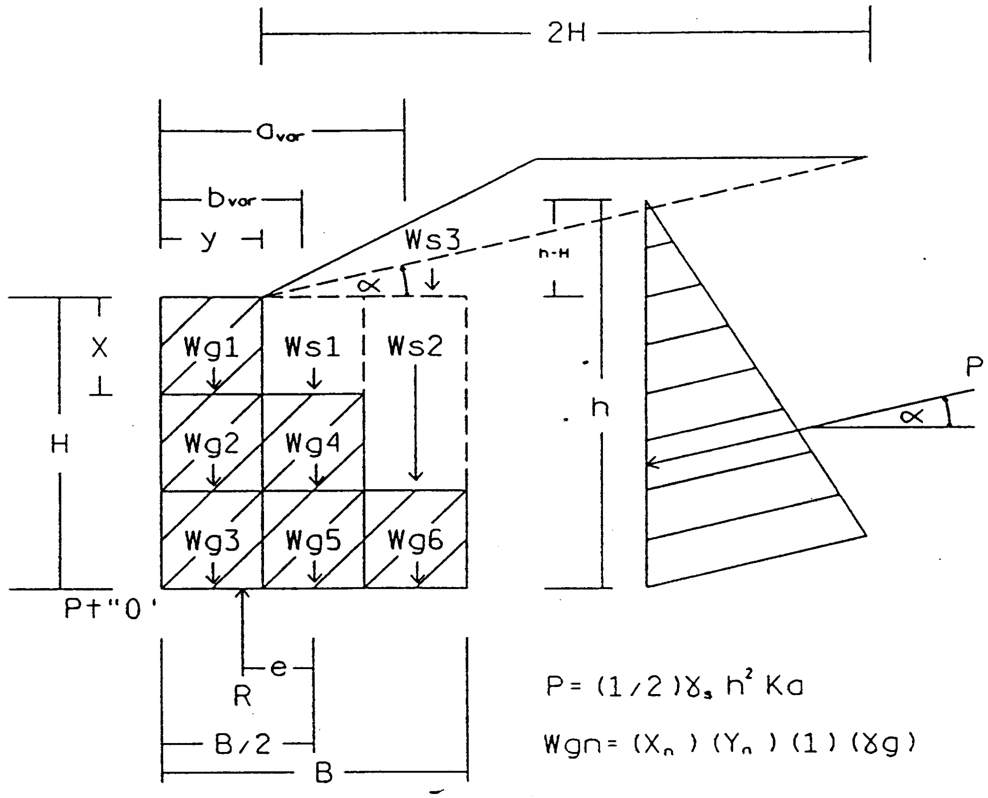
where b = distance to gabion section center of gravity
a = distance to soil block center of gravity

Bearing Pressure

Base Pressure $(\sigma_v) = \dfrac{R}{B-2e}$ = q allowable (allowable soil bearing capacity)

where R = Σ Wgn + Σ Wsn, e = eccentricity $= \dfrac{B}{2} - \dfrac{M_R - M_D}{R}$

FIGURE 14.9
Broken Back Slope



Safety Factor Against Sliding

$$\text{S.F.} = \frac{\text{Resisting Forces}}{\text{Driving Forces}} = \frac{F_R}{F_D} = \frac{[\sum Wgn + \sum Wsn] + P\sin\alpha]\ \text{Tan}\ \phi}{P\ \text{Cos}\ \alpha}\ {}^{3}\ 1.5$$

Safety Factor Against Rotation about Pt. "0'"

$$\text{S.F.} = \frac{\text{Resisting Moments}}{\text{Driving Moments}} = \frac{M_R}{M_D} = \frac{\sum Wgn\ (b_n) + \sum Wsn\ (a_n) + P\sin\alpha\ (B)}{P\cos\alpha\ (h/3)} \geq 2.0$$

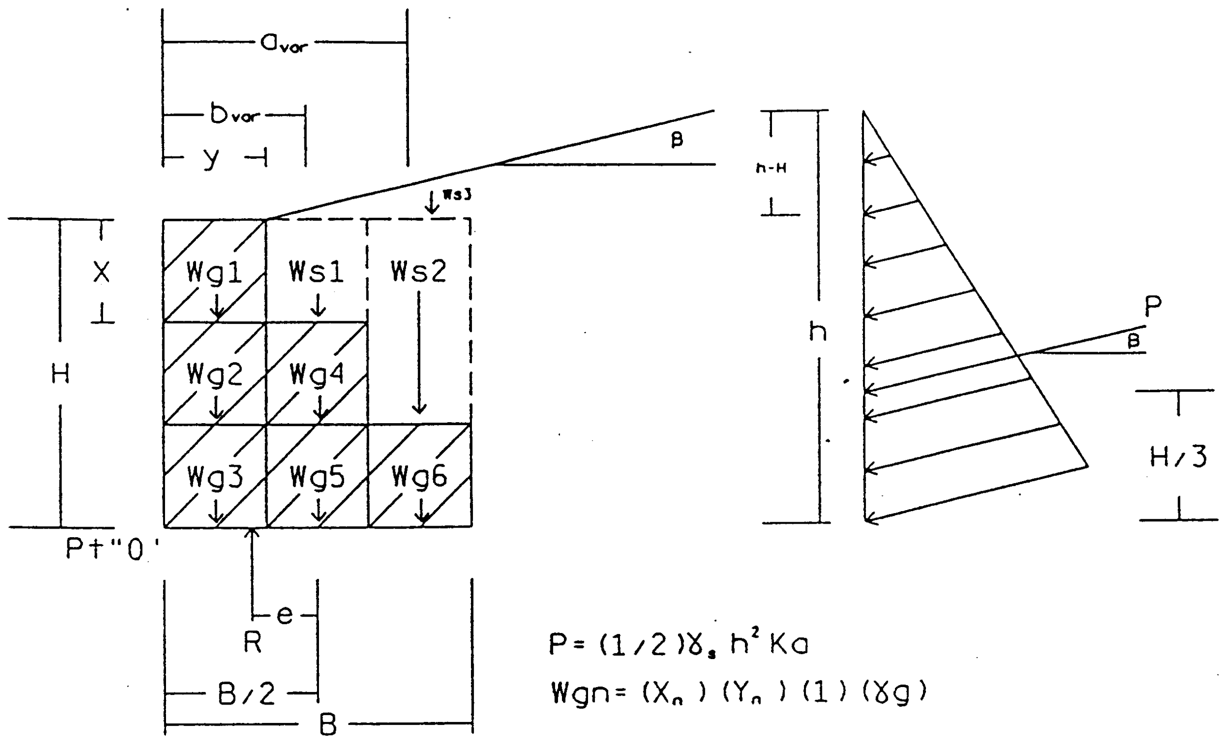
where b_n = distance to gabion section center of gravity
a_n = distance to soil block center of gravity

Bearing Pressure

Base Pressure (σ_v) $= \dfrac{R}{B-2e}$ = q allowable (allowable soil bearing capacity)

where R = S Wgn + Wsn + P sin α, e = eccentricity = $\dfrac{B}{2} - \dfrac{M_R - M_D}{R}$

FIGURE 14.10
Sloping Backfill



Safety Factor Against Sliding

$$\text{S.F.} = \frac{\text{Resisting Forces}}{\text{Driving Forces}} = \frac{F_R}{F_D} = \frac{[\Sigma\ \text{Wgn} + \Sigma\ \text{Wsn}] + \text{Psin}\beta]\ \text{Tan}\ \phi}{\text{P Cos}\beta} = 1.5$$

Safety Factor Against Rotation about Pt. "0'"

$$\text{S.F.} = \frac{\text{Resisting Moments}}{\text{Driving Moments}} = \frac{M_R}{M_D} = \frac{\Sigma\ \text{Wgn}\ (b_n) + \Sigma\ \text{Wgn}\ (a_n) + \text{Psin}\beta(\text{B})}{\text{P cos}\beta\ (h/3)} = 2.0$$

where b_n = distance to gabion section center of gravity
a_n = distance to soil block center of gravity

Bearing Pressure

Base Pressure (σ_v) = $\frac{R}{B-2e}$ = q allowable (allowable soil bearing capacity)

where R = Σ Wgn + Σ Wsn + P sinß, e = eccentricity = $\frac{B}{2} - \frac{M_R - M_D}{R}$

(2) Summary of Design Safety Factors and Requirements

a. Safety Factors

Overturning	=	2.0
Sliding	=	1.5
Global	=	1.3

b. Foundation Design Parameters

Use values provided by WisDOT.

c. Traffic Surcharge

Traffic live load surcharge = 2 feet = 240 lb/ft^2

d. Retained Soil

Unit weight = 120 lb/ft^3
Angle of internal friction as determined from tests from WisDOT.

e. Soil Pressure Theory

Rankine's Theory or Coulomb Theory, at the discretion of the designer.

14.3(D) STEEL SHEET PILING WALLS

Steel sheet piling walls are normally used as temporary walls but they can also be used for permanent locations.

(1) Design Procedure for Sheet Piling Walls

A description of their design along with some simplified earth pressure distributions are given in AASHTO "Standard Specifications for Highway Bridges" Article 5.6, "Nongravity Cantilevered Wall Design". They are also referred to as flexible cantilevered walls. Steel sheet pile walls can be designed as cantilevered walls up to approximately 15 feet in height. Over 15 feet steel sheet pile walls may require tie-backs with either prestressed soil anchors or deadman type anchors. The preferred method of designing cantilever sheet piling is by the "Conventional Method" as described in "United States Steel Sheet Piling Design Manual", February, 1974. WisDOT provides the soil design parameters including cohesion values, angles of internal friction, angles of wall friction, soil densities, and water table elevations.

Anchored wall design must be in accordance with AASHTO "Standard Specifications for Highway Bridges" Article 5.7. Anchors for permanent walls shall be fully encapsulated over their entire length. The anchor hardware shall be designed to have a corrosion resistance durability to ensure a minimum design life of 75 years.

All areas of permanent steel sheet piling above the ground line shall be coated or painted prior to driving, or shall be made from weathering steel. Corrosion potential should be considered in all steel sheet piling designs. Special consideration should be given to permanent steel sheet piling used in areas of Northern Wisconsin which are inhabited by corrosion causing bacteria (see FDM Procedure 13-1-15).

The appearance of permanent steel sheet piling walls may be enhanced by applying either precast concrete panels or cast-in-place concrete surfacing. Welded stud-shear connectors can be used to attach cast-in-place concrete to the sheet piling. Special surface finishes obtained by using form liners or other means and concrete stain or a combination of stain and paint are recommended for the concrete facing.

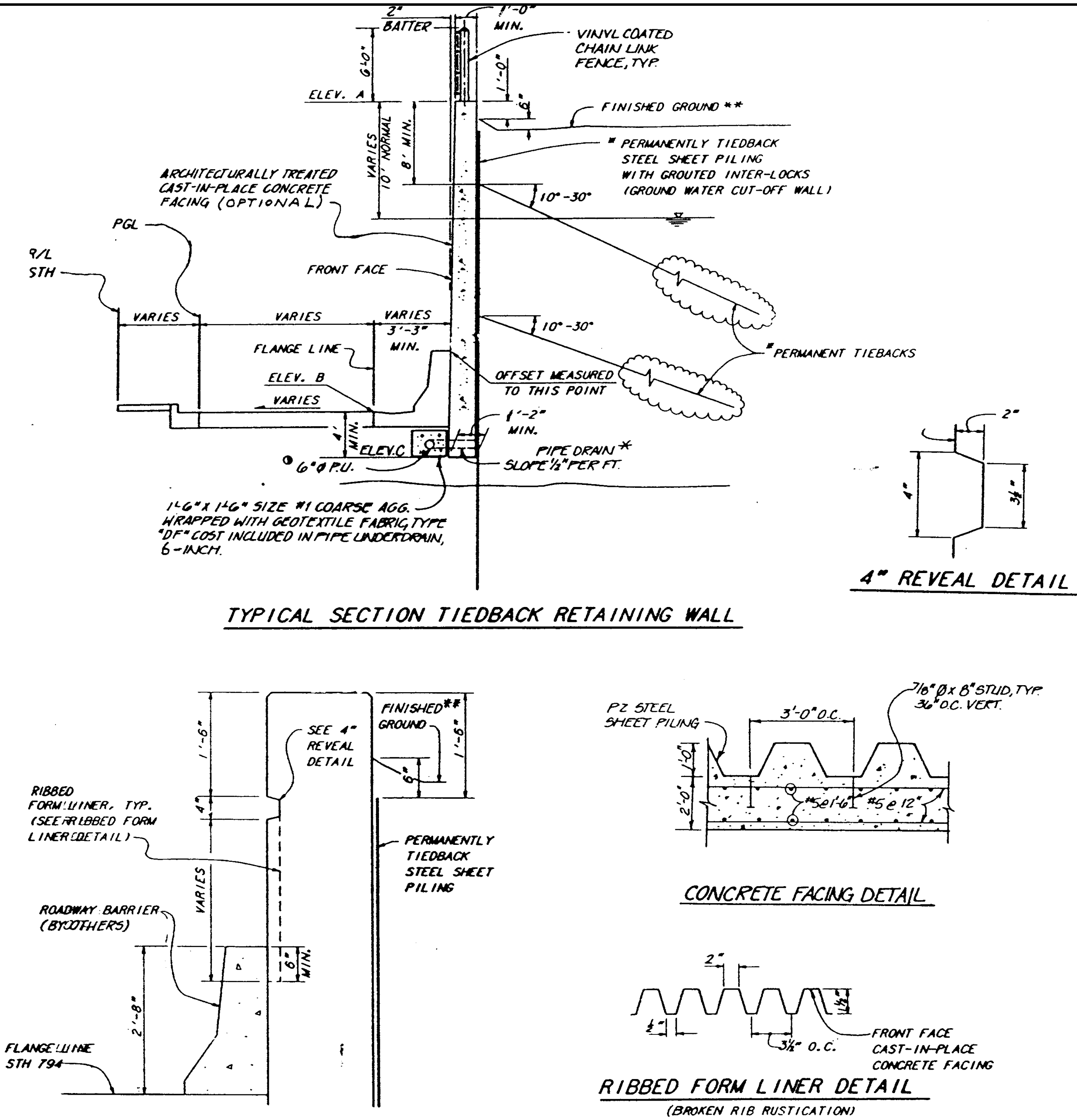


FIGURE 14.11

EXAMPLE OF CONCEPTUAL DESIGN OF ANCHORED SHEET PILE WALL

(2) Summary of Design Safety Factors and Requirements

a. Safety Factors

Global $\geq$ 1.3
Passive Pressure Reduction Kp' = 2/3*Kp

b. Foundation Design Parameters

Use values provided by WisDOT.

c. Traffic Surcharge

| Traffic live load surcharge = 2 feet = 240 lb/ft^2 or determined by site
| condition.

d. Retained Soil

Unit weight = 120 lb/ft^3
Angle of internal friction as determined from tests from WisDOT.

e. Soil Pressure Theory

| Coulomb Theory.

f. Design Life for Anchorage Hardware

75 years minimum

g. Steel Design Properties

Minimum yield strength = 39,000 psi

14.3(E) MODULAR BLOCK GRAVITY WALLS

The proprietary modular blocks used in combination with soil reinforcements "Mechanically Stabilized Earth Retaining Walls With Modular Block Facings" can also be used as pure gravity walls (no soil reinforcement). The height to which they can be constructed is a function of the depth of the blocks, the setback of the blocks, the backslope angle, and the angles of internal friction of the retained soil behind the wall. The minimum embedment to the top of the footing for Block Gravity Walls is the same as stated in AASHTO LRFD 11.10.2.2 for mechanically stabilized earth walls. Walls of this type are limited to exposed heights of 4.33 feet or less, and are limited to a maximum differential settlement of 1/200. (Exceptions for greater heights should be submitted to Structures Development for approval, 608-266-8494).

Footings for Block Gravity Walls are either base aggregate dense 1 ¼ inch (section 305 of the standard specifications) or Grade A concrete. Minimum footing thickness is 12 inches for aggregate and 6 inches for concrete. The width of the footing equals the width of the bottom block plus 12 inches for aggregate footings and plus 6 inches for concrete footings. The extra footing width extends beyond the front face of the wall. The coarse aggregate No. 1 (501.2.5.4 of the standard specifications), which is placed within 1 foot behind the back face of the wall, extends down to the bottom of the footing. The standard special provisions for Modular Block Gravity Walls require a concrete footing if any portion of a wall is over 5 feet measured from the top of the footing to the bottom of the wall cap.

The material specifications for the blocks used for gravity walls are identical to those for the blocks used for block MSE walls.

The design of Modular Block Gravity Walls provided by the wall supplier must be in compliance with the WisDOT special provisions for the project and the policy and procedures as stated in the WisDOT Bridge Manual. The design must include an analysis of external stability including overturning, sliding, and soil bearing stress. Horizontal shear capacity between blocks must also be verified. Global analysis and settlement calculations and allowable bearing capacity are the responsibility of the WisDOT. The soil design parameters to use for the design are provided by WisDOT including the minimum block depth allowed. Design drawings and calculations must be submitted to WisDOT for approval. Computer output from automated designs may be submitted in lieu of manual design calculations if approved by WisDOT.

(1) Design Procedure for Modular Block Gravity Walls

When designing a “Modular Block Gravity Wall” without setback and with level backfill the active earth pressure coefficient may be determined from the following formula.

$$Ka = \tan^2(45 - \phi_r/2)$$

When designing a "Modular Block Gravity Wall" with setback the active earth pressure coefficient Ka shall be determined from the following Coulomb formula. The interface friction angle between the blocks and soil behind the blocks is assumed to be zero.

$$Ka = \frac{\cos^2(\phi_r + A)}{\cos^2 A \cos A \ (1 \ + \ (Z/Y)^{1/2})^2}$$

$$Z = \sin \phi_r \sin(\phi_r - B)$$

$$Y = \cos A \cos(A+B)$$

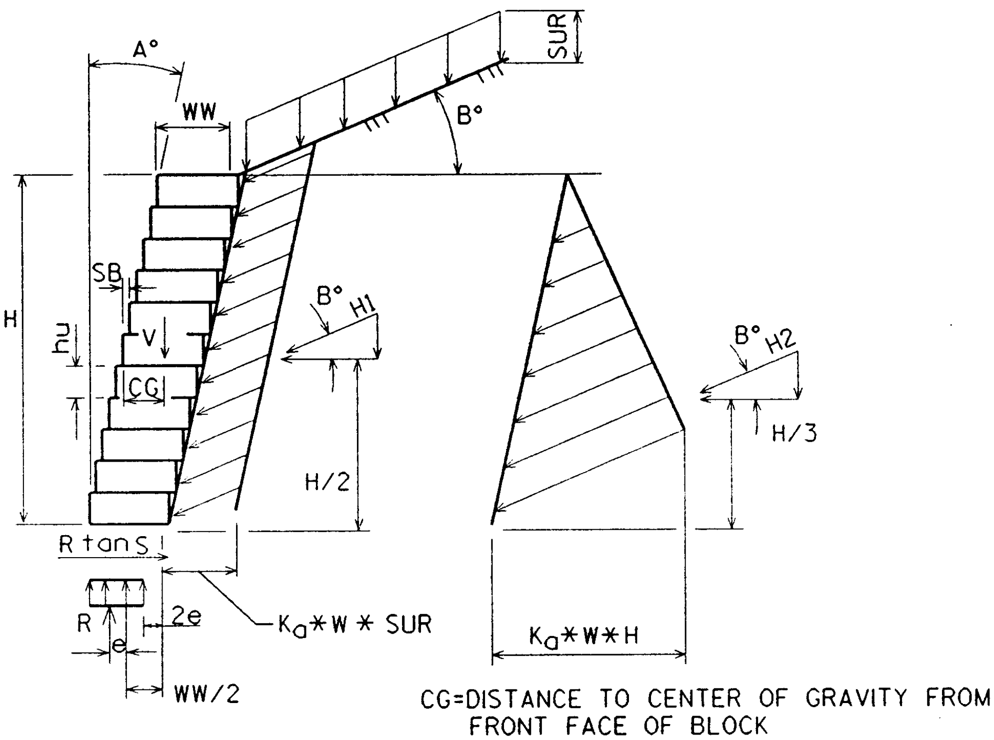


FIGURE 14.12

MODULAR BLOCK GRAVITY WALL ANALYSIS

The forces acting on a modular block gravity wall are shown in Figure 14.12. The density of the blocks are assumed to be 135 pcf and the drainage aggregate inside or between the blocks 120 pcf. No passive soil pressure is allowed to resist sliding. Shear between the blocks must be resisted by friction, keys or pins. The soil pressure on the bottom block may be computed by using the Meyerhof distribution method.

Factor of safety against overturning

For overturning, moments are taken about the outside corner of the bottom block. The equivalent weight of the blocks and drainage aggregate in pounds/cubic foot = w. The center of gravity of the block and infill is assumed to be at ww/2 from the front face for the following calculations.

The resisting moment is equal to:

H*WW*w*[(H-hu)*.5*tanA +CG/2]

The overturning moment is equal to:

H1*cosB*H/2 + H2*cosB*H/3, where H1=Ka*W_r*sur*H and H2= Ka*W_r*H^2/2

FS overturning = resisting moment/overturning moment.

FS overturning must be equal to or greater than 2.

Note: The vertical components of H1 and H2 are conservatively ignored.

Factor of safety against sliding

R tanS/cosB(H1+H2) must be equal to or greater than 1.5, where tanS is the coefficient of sliding friction (tanS to be provided by WisDOT) and R=V=summation of the vertical loads.

V=H*WW*w

Bearing pressure

The bearing pressure at the bottom of the lower block using the Meyerhof stress distribution equals:

BP = R/(WW-2*e)

e is calculated by taking moments about the center of the lower block

e = [H1cosB°*H/2+(H2cosB°*H/3)-V*(H-hu)*.5*tanA]/R

Note: If e is negative, the maximum soil pressure is at the inside face (backfill side) of the block. Use a positive e in the equation for BP.

BP ≤ "allowable bearing capacity" which is provided by the WisDOT.

(2) Summary of Design Safety Factors and Requirements

a. Safety Factors

Overturning ≥ 2.0
Sliding≥ 1.5
Global ≥ 1.3

b. Block Data

One piece block.
Minimum thickness of front face = 4 inches.
Minimum thickness of internal cavity walls other than front face = 2 inches.
28 day concrete strength = 5000 psi. Maximum water absorption rate by weight = 5%.
Sealer required on walls with significant exposure to salt.

c. Traffic Surcharge

Traffic live load surcharge = 2 feet = 240 lb/ft^2.

d. Retained Soil

Unit weight = 120 lb/ft^3
Angle of internal friction as determined from tests from WisDOT.

e. Soil Pressure Theory

For analysis of the wall use Coulomb Theory.

| f. Maximum Height = 5.33’

| (This height is measured from top of footing to bottom of cap. It is not the
| exposed height).
| In addition this height may be reduced if there is sloping backfill or a sloping
| surface in front of the wall.

| g. See Appendix B for discussion of gravity block walls and MSE block walls.

14.3(F) MECHANICALLY STABILIZED EARTH RETAINING WALLS GENERAL DISCUSSION

Mechanically Stabilized Earth (MSE) is the term used to describe the practice of reinforcing a mass of soil with either metallic or geosynthetic soil reinforcement which allows the mass of soil to function as a gravity retaining wall structure. The soil reinforcement is placed horizontally across potential planes of shear failure and develops tension stresses to keep the soil mass intact. The soil reinforcement is attached to a facing at the front face. The behavior of the mechanically stabilized earth and facing is analogous to how a reinforced block of concrete equal in section to the wall height and the length of the soil reinforcement would function to retain the same earth loads.

MSE walls are proprietary wall systems and the structural design of the wall system is provided by the wall supplier. Design drawings and calculations must be submitted to WisDOT for acceptance. Computer output from automated designs may be submitted in lieu of manual design calculations if approved by WisDOT. Horizontal alignment or limits of horizontal alignment and grades at the bottom and top of the wall are determined by the Wisconsin Department of Transportation (WisDOT). The design must be in compliance with the WisDOT special provisions for the project and the policy and procedures as stated in the WisDOT Bridge Manual. Walls adjacent to roadways shall be designed for a liveload surcharge of 240 lbs/square foot when traffic can come within 1/2 the height of the wall from the backface of the reinforced soil mass. The minimum embedment of the wall to the top of the leveling pad shall be as stated in AASHTO "Standard Specifications for Highway Bridges", Article 5.8.1.

The design of the MSE wall system by the wall supplier must include an analysis of internal stability (soil reinforcement pullout and stress) and local stability (facing connection forces and internal panel stresses). The wall supplier is not required to perform an external stability analysis or a global analysis (overall slope stability). Global analysis, external stability (overturning, soil bearing stress, and sliding), and settlement calculations are the responsibility of the WisDOT. Soil bearing stress and base sliding are checked by WisDOT during the wall selection process. Soil borings and soil design parameters are provided by WisDOT.

MSE walls do not require footings because the wall facing and the mechanically stabilized earth behind it act as an integral unit. Soil pressures from the vertical and horizontal earth loads are distributed over almost the entire length of the reinforced soil zone. It is important that the mechanically stabilized earth and wall facing be allowed to settle equally. Piling or a rigid footing under the wall should not be used. A 6” (150 mm) unreinforced concrete leveling pad foundation is required for all panel wall facings and all block wall facings over 5 feet(1.5 m) in height.

MSE walls can be designed to accept differential vertical movement. Joints to accommodate differential vertical movement are required when MSE walls abut rigid structures or when differential movement of more than 1% is anticipated between wall segments. Such joints should also be included at any point where an abrupt change of vertical movement is anticipated. An example of this condition would be at the location of the outer walls of a concrete box culvert passing through the wall.

The distribution of soil pressure at the bottom of the reinforced soil zone, or at any horizontal plane above it, from vertical and horizontal loads may assumed to be uniform over a length determined from the Meyerhof method. The Meyerhof method is a simplified procedure for calculating a uniform pressure which is representative of the actual variable vertical soil pressure.

Soil reinforcement can be either metallic (strips or bar grids like welded wire fabric) or geogrids (polyester, polypropylene, or high density polyethylene). Wall facings used on DOT projects must be preapproved. The galvanized steel reinforcement that is used for soil reinforcement is oversized in cross sectional areas to account for the corrosion that occurs during the life of the structure and the resulting loss of section. The net section remaining after corrosion at the end of the design service life is used to check allowable stress. MSE walls are designed for a minimum service life of 75 years. Corrosion of the wall anchors that connect the soil reinforcement to the wall must also be accounted for in the design.

(1) Usage Restrictions for MSE Walls

MSE walls with either block or panel facings cannot be used if any of the following conditions exist.

(a) External stability requirements can not be satisfied.
(b) The available construction limit behind the wall is less than .7 times the wall height or 6'-0, whichever is greater, plus 1'-6, plus an additional distance based on OSHA requirements. If tiered walls are used the affects to each level must be included.
(c) Utilities subject to emergency repair can only be located within the reinforced soil zone and can only be repaired by excavating thru the reinforced soil zone.

In addition to the above MSE walls with block facings cannot be used if any of the following conditions exist.

(a) The wall is a component of an abutment structure, either a wingwall or a wall parallel to the C/L of bearing.
(b) Traffic barriers or roadway pavements are vertically supported by the wall.
(c) Differential settlement exceeds 1 in 200.
(d) If a railing or fence is placed behind the wall, the posts cannot be driven thru the geogrids as it will misalign the facing. The grid must be cut and holes made or preferably a blockout is provided. This only applies to the top layer which should be no more than 24 inches below the top of the block finished grade. Penetration thru the second layer of geogrid is not allowed.

14.3(G) MECHANICALLY STABILIZED EARTH RETAINING WALLS WITH PRECAST CONCRETE PANEL FACINGS

The geometric pattern of the joints and the smooth uniform surface finish of the factory provided panels gives them an aesthetically pleasing appearance. They are the required MSE wall system to use when supporting traffic live loads which are in close proximity to the wall. They are also allowed as components of an abutment structure. Either steel strips or welded wire fabric is allowed for soil reinforcement.

Although abutment loads can be supported on spread footings within the reinforced soil zone, it is the policy of WisDOT to support the abutment loads for multiple span structures on piles or shafts that pass through the reinforced soil zone to the insitu soil below. Piles shall be driven or shafts placed prior to the placement of the reinforced earth. Strip type reinforcement can be skewed around piles but must be connected to the wall panels and must extend to the rear of the reinforced soil zone. For continuous welded wire fabric reinforcement, the supplier should provide details on the plans showing how to allow placement around piles or any other obstacle. Abutments for single span structures may be supported by spread footings placed within the soil reinforcing zone. Loads from such footings must be considered for both internal wall design and external stability considerations.

Wall panels are available in a plain concrete finish or numerous form liner finishes and textures. An exposed aggregate finish is also available along with earth tone colors. Although color can be obtained by adding additives to the concrete mix it is more economical to obtain color by applying concrete stain and/or paint at the job site.

The allowable differential settlement in a length of MSE walls as a function of joint width is given in AASHTO "Standard Specifications for Highway Bridges" Article 5.2.1.4. Alignment pins control horizontal spacing of panels and a vertical space of 3/4" will allow 1% differential settlement without causing concrete to concrete contact. Horizontal panel joints are formed by using compressible joint material. All joints between wall panels are covered on the backside of the wall with geotextile fabric, Type DF, to prevent backfill material from leaking thru the wall but allowing moisture to pass thru. Slip joints are used to handle differential settlement in excess of 1%. Slip joints should also be included at any point of anticipated abrupt changes in vertical movement.

(1) <u>Design Procedure for MSE Vertical Face Walls with Metallic (Inextensible) Soil Reinforcement</u>

The Design Procedure for 3 Cases is presented:

Case A -- Horizontal Backslope (B=0°)
Case B -- Sloping Backfill (B>0°)
Case C -- Broken Back Backfill (B>0°)

The active earth pressure coefficient for external stability can be determined from the following formula:

$$Ka = COS\ B * (COS\ B - X)/(COS\ B + X)$$

where B = slope angle of backfill behind wall
ϕ_r = angle of internal friction of retained soil
X = SQRT (COS^2 B - COS^2 f_r)

The actual angle of internal friction of the retained soil which is supplied by WisDOT shall be used. If the contractor uses temporary sheet piling or shoring to support the excavation then the angle of internal friction of the in-situ soil is used for external stability calculations. If the in-situ soil is removed to a stable slope then the angle of internal friction of the backfill material that replaces the excavated material is used.

Ka values are given in the following table.

Ka = Active Earth Pressure Coefficients (Rankine)

B (degrees)	0	5	10	15	20	25	27
ϕ_r = 15°	.589	.605	.664	.966	---	---	---
ϕ_r = 20°	.490	.500	.531	.603	.940	---	---
ϕ_r = 25°	.406	.412	.431	.469	.547	.906	---
ϕ_r = 30°	.333	.337	.350	.373	.414	.494	.552
ϕ_r = 34°	.283	.286	.294	.311	.338	.385	.413

Figure 14.13 shows how to calculate external stability for a horizontal backslope and 2 foot live load surcharge.

Case A: Horizontal Backslope (B=0°)

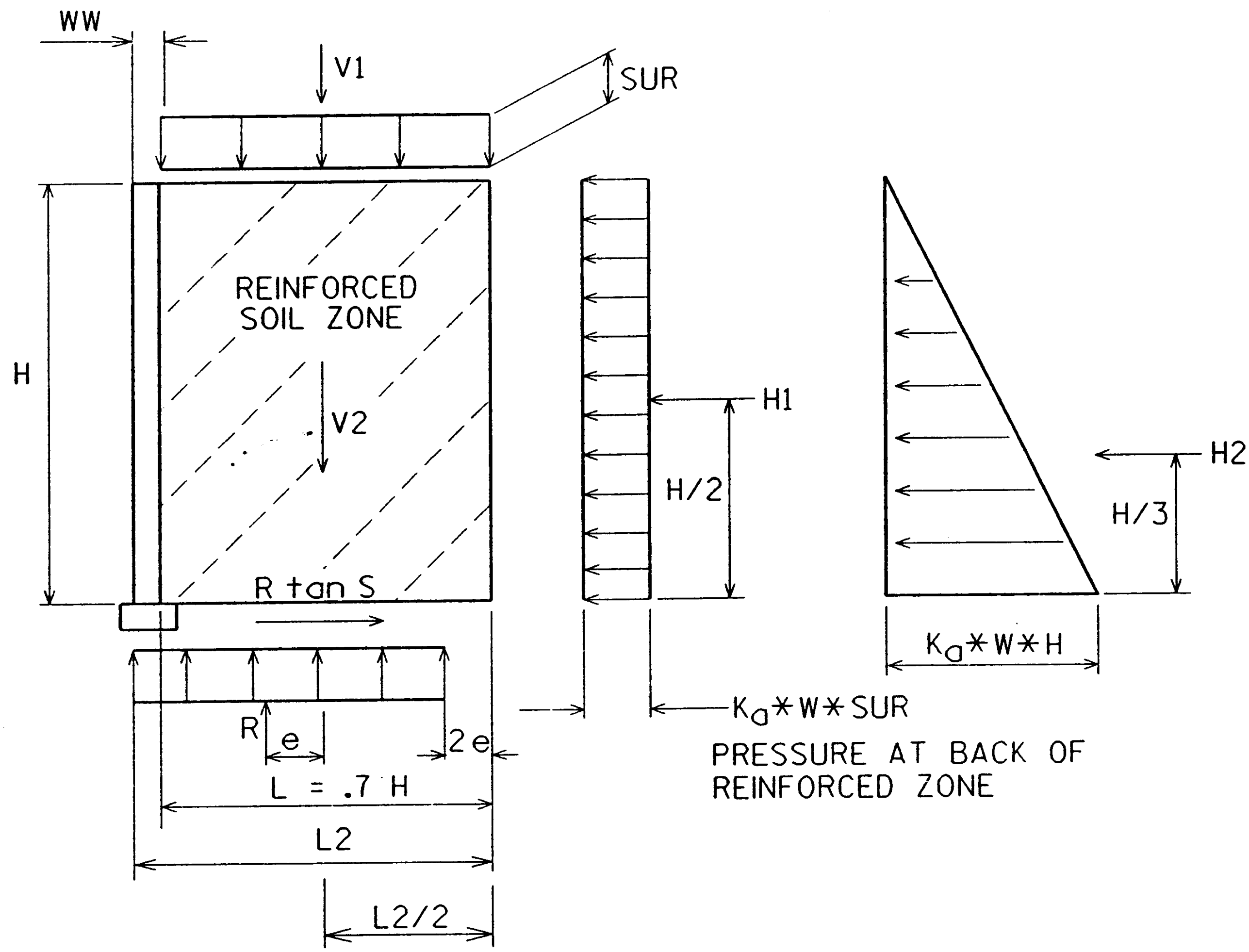


FIGURE 14.13

L = length of soil reinf. (ft.)
Wr = (unit weight of soil) for retained soil (kip/cubic foot)
W_i = .12 kip/cubic foot (unit weight of soil) for reinf. infill soil
V1 = $SUR*W_i * L$
V2 = $W_i * H*L2$
H1 = Ka*0.12*SUR*H
H2 = $Ka*W_r*H*H/2$
R = V1 + V2
Ka = see formula on previous page
tanS = coeff. of friction

Factor of safety against overturning = $\sum$ Resisting Moments/$\sum$ Overturning Moments ≥ 2.0

$$V2(L2/2)/(H1(H/2)+H2(H/3)) \geq 2.$$

Note: Surcharge is assumed to act outside of the reinforced soil mass for overturning and sliding analysis and therefore V1 is not included. V1 is used to compute maximum bearing pressure.

Factor of safety against sliding friction = $\sum$ Resisting Forces/$\sum$ Driving Forces ≥ 1.5

$$V2 \tan S/(H1+H2) \geq 1.5$$

where tan S is the coefficient of sliding friction. (tan S to be provided by WisDOT).

The bearing pressure at the bottom of the reinforced soil mass = BP (kips/ft^2)

$$BP = R/(L2-(2*e)) \text{ where}$$
$$e = (H1*H/2 + H2*H/3 - WW/2*V1)/R$$

The "allowable bearing capacity" which is provided by the WisDOT must be greater than or equal to "BP".

Figure 14.14 shows how to calculate external stability for a sloping backfill case.

CASE B: Sloping Backfill (B>0°)

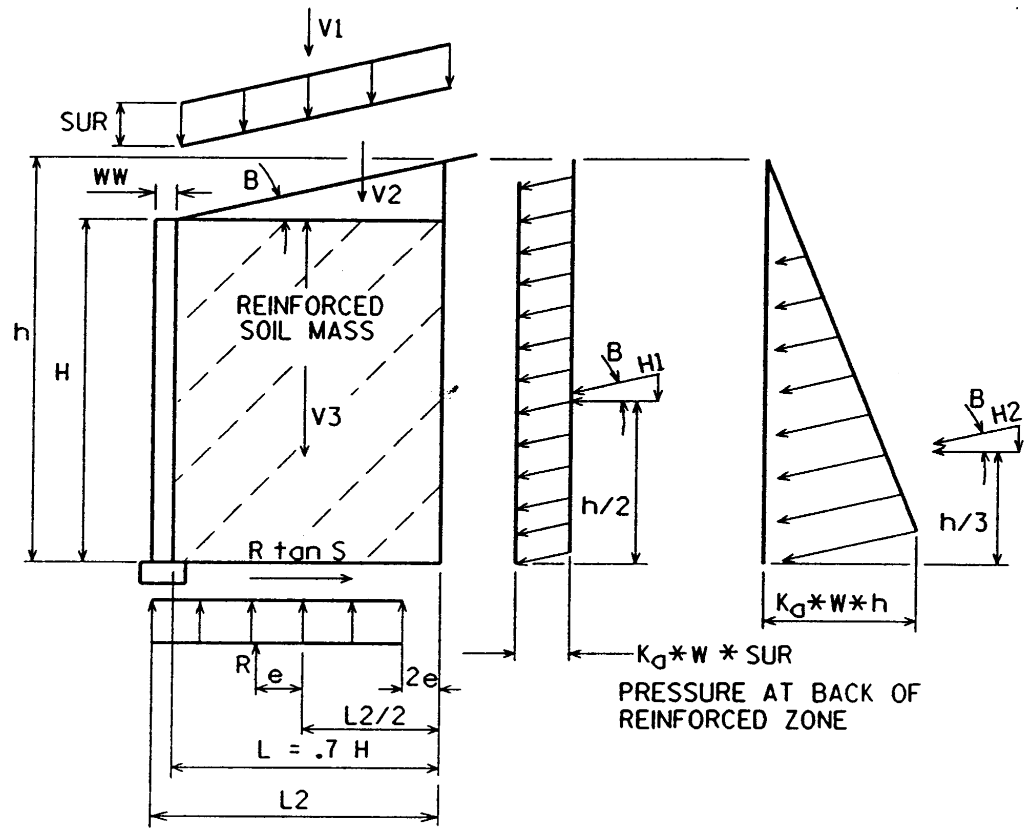


FIGURE 14.14

L = length of soil reinf. (ft)
W_r = (unit weight of soil) for retained soil (kips/ft^2)
W_i = .12 kip/cubic foot (unit weight of soil) for reinf. infill soil
$V1 = SUR*W_i*L$
$h = H+L*\tan(B)$
$V2 = W_i*(h-H)*L/2$
$V3 = W_i*H*L2$
$H1 = SUR*Ka*0.12*h$
$H2 = Ka*W_r*h*h/2$
$R = V1 + V2 + V3 + \sin B(H1+H2)$
Ka = see formula @ beginning of design procedure
tanS = coeff. of friction

Factor of safety against overturning = $\sum$ Resisting Moments/$\sum$ Overturning Moments ≥ 2.0

$$\frac{V3*L2/2 + V2*(2/3L+WW) + L2*\sin B(H1+H2)}{H1*\cos B*h/2 + H2*\cos B*h/3} \geq 2.$$

Note: For overturning and sliding analysis, surcharge is assumed to act outside of the reinforced soil zone and therefore V1 is not used. V1 is used to compute maximum bearing pressure.

Factor of safety against sliding friction = $\sum$ Resisting Forces/S Driving Forces ≥ 1.5

$$(V2+V3)\tan S/\cos B(H1+H2) \geq 1.5$$

where tanS is coefficient of sliding friction. (tanS to be provided by WisDOT).

The bearing pressure at the bottom of the reinforced soil mass = BP (kips/ft^2)

$$BP = R/(L2-(2*e)) \quad \text{where}$$

$$e = \left(H1*\cos B*h/2 + H2*\cos B*h/3 - \sin B(H1+H2)L2/2 - V1\left(\frac{WW}{2}\right) - V2\left(\frac{L2}{6} + \frac{WW}{3}\right)\right)/R$$

Note: e is determined by taking moments about center of base length L2.

The "allowable bearing capacity" which is provided by the WisDOT must be greater than or equal to "BP".

Figure 14.15 shows how to calculate external stability for a broken back backfill case.

CASE C: Broken Back Backfill (B>0°)

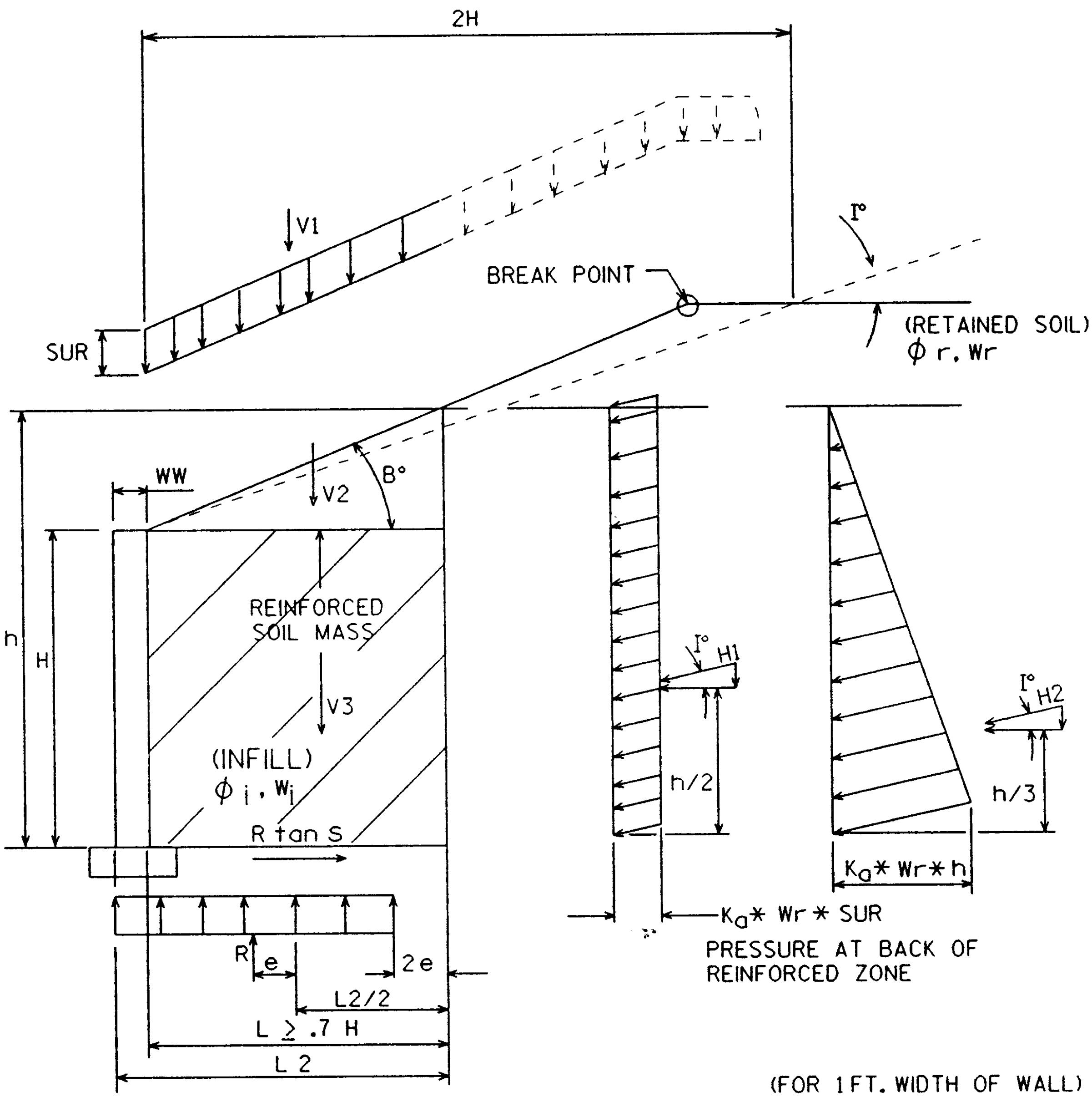


FIGURE 14.15

If a break in the slope behind the wall facing is located horizontally within two times the height of the wall 2H, a broken back slope design CASE C may be used. If the break is located at 2H or more from the wall facing, CASE B is to be used.

The only difference between CASE C and CASE B is the magnitude and direction for forces H1 and H2. The magnitude of these forces is a function of the active earth pressure coefficient Ka which is shown at beginning of design procedure. CASE C and CASE B both use this formula for Ka, but for CASE C substitute I° for B° in the formula. In CASE C these forces make an angle of I° with the horizontal as shown in Figure 14.15 and for CASE B they make an angle of B° with the horizontal as shown in Figure 14.14.

When the break in the slope behind the wall is located horizontally within 0.5H from the backface of the reinforced soil mass, where H is the height of the wall, then liveload surcharge is considered as part of the design. If the break is located at 0.5H or more from the backface of the reinforced soil mass, liveload surcharge is not considered as part of the design.

L = length of soil reinf. (ft.)
Wr = (unit of weight of soil) for retained soil. (Kips/cubic foot)
Wi = 0.12 Kip/cubic foot (unit weight of soil) for reinf. infill soil
V1 = SUR*Wi*L (Kips)
V2 = Wi(h-H)*L/2 (Kips)
V3 = Wi*H*L2 (Kips)
H1 = SUR*Ka*0.12*h (Kips)
H2 = Ka*Wr*h*h/2 (Kips)
R = V1+V2+V3+SIN I(H1+H2) (Kips)
Ka = see formula at beginning of design procedure. (But replace B° with I°)
tan S = coefficient of friction

Analysis of Overturning - AASHTO 5.5.5

Factor of safety against overturning = ∑ Resisting Moments/∑ Overturning Moments ≥ 2.0

$$\frac{V3*L2/2 + V2*(2/3L+WW) + L2*\sin I(H1+H2)}{H1\cos I*h/2 + H2*\cos I*h/3} \geq 2.0$$

Note: For overturning and sliding analysis, surcharge is assumed to act outside of the reinforced soil zone and therefore V1 is not used. V1 is used to compute maximum bearing pressure.

Analysis of Sliding - AASHTO 5.5.5

Factor of safety against sliding = ∑ Resisting Forces/∑ Driving Forces ≥ 1.5

$$(V2+V3)\tan S/\cos I(H1+H2) \geq 1.5$$

where tanS is coefficient of sliding friction. (tanS to be provided by WisDOT).

<u>Analysis of Soil Bearing Pressure</u> - AASHTO 5.8.3

The bearing pressure at the bottom of the reinforced soil mass = BP(Kips/ft^2.)

BP = R/(L2-(2*e)) where

$$e = \left(H1 \cdot \cos I \cdot h/2 + H2 \cdot \cos I \cdot h/3 - \sin I(H1+H2)L2/2 - V1\left(\frac{WW}{2}\right) - V2\left(\frac{L2}{6}+\frac{WW}{3}\right)\right)/R$$

Note: e is determined by taking moments about center of base length L2.

The "allowable bearing capacity" which is provided by the WisDOT must be greater than or equal to "BP".

The previous equations for "h, V2, Factor of Safety against Overturning and Bearing Pressure" in CASE C are correct when the breakpoint of the slope is greater than or equal to "L" from the backface of the wall as shown in Figure 14.15. If the breakpoint is less than "L" away, then these equations will have to be modified.

The "failure plane" for MSE walls with metallic strip or grid reinforcements (inextensible) is shown in Figure 14.16. The procedure to calculate stress at the failure plane and at the face of the wall follows.

The horizontal stress σ_h at each reinforcement level shall be computed by multiplying the vertical stress σ_v at that level by an earth pressure coefficient, K, as shown in Figure 14.16. The vertical stress σ_v is based on the vertical loads being distributed over a length determined by the Meyerhof formula.

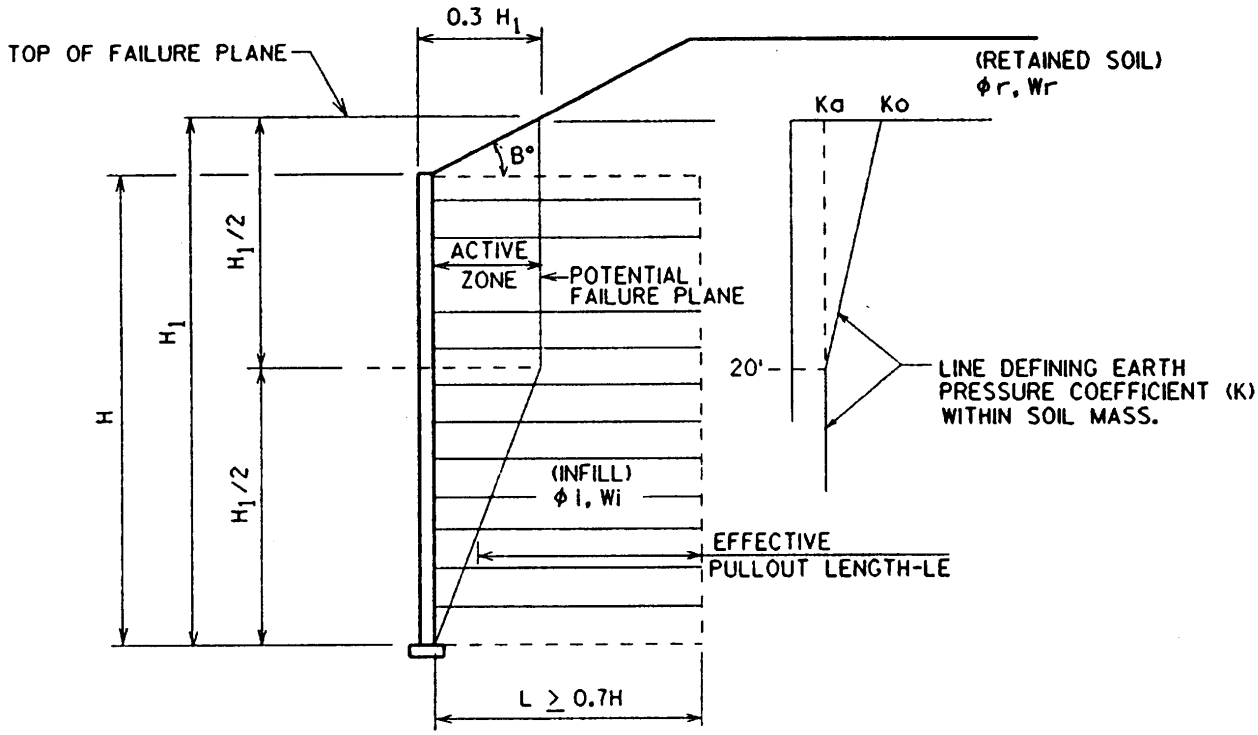
The top of the failure plane is located H_1 from the top of the leveling pad.

CASE A - Horizontal Backslope (B=0°): $H_1 = H$

CASE B - Sloping Backfill (B>0°) $H_1 = H + \dfrac{\text{Tan B}*0.3H}{(1-0.3\text{ Tan B})}$

CASE C - Broken Back Backfill (B>0°) $H_1 = H + \dfrac{\text{Tan B}*0.3H}{(1-0.3\text{ Tan B})}$

CASE A, B and C are depicted in Figures 14.13, 14.14 and 14.15 respectively.



Ka = TAN^2 (45-ϕi/2) - - (ACTIVE EARTH PRESSURE COEFF.)
Ko = 1-SIN ϕi - - - - - - - (AT-REST EARTH PRESSURE COEFF.)
ϕi = ANGLE OF INTERNAL FRICTION OF INFILL SOIL
ϕi = 34° FROM TESTS
L = LENGTH OF SOIL REINFORCEMENT

DETERMINATION OF FAILURE PLANE LOCATION AND EARTH COEFFICIENTS FOR INEXTENSIBLE REINFORCEMENTS.

(FOR 1 FT. WIDTH OF WALL)

FIGURE 14.16

K decreases linearly from the top of the intersection of the failure plane with the ground surface to an active coefficient of earth pressure Ka at 20 feet below. Beyond 20 feet Ka shall be used. This design procedure assumes Ko and Ka remain the same regardless of the external loading conditions. This assumption is based on the results of field measurements.

The force in the soil reinforcement is determined at the location of the failure plane as follows:

CASE A: HORIZONTAL BACKSLOPE (B=0°)

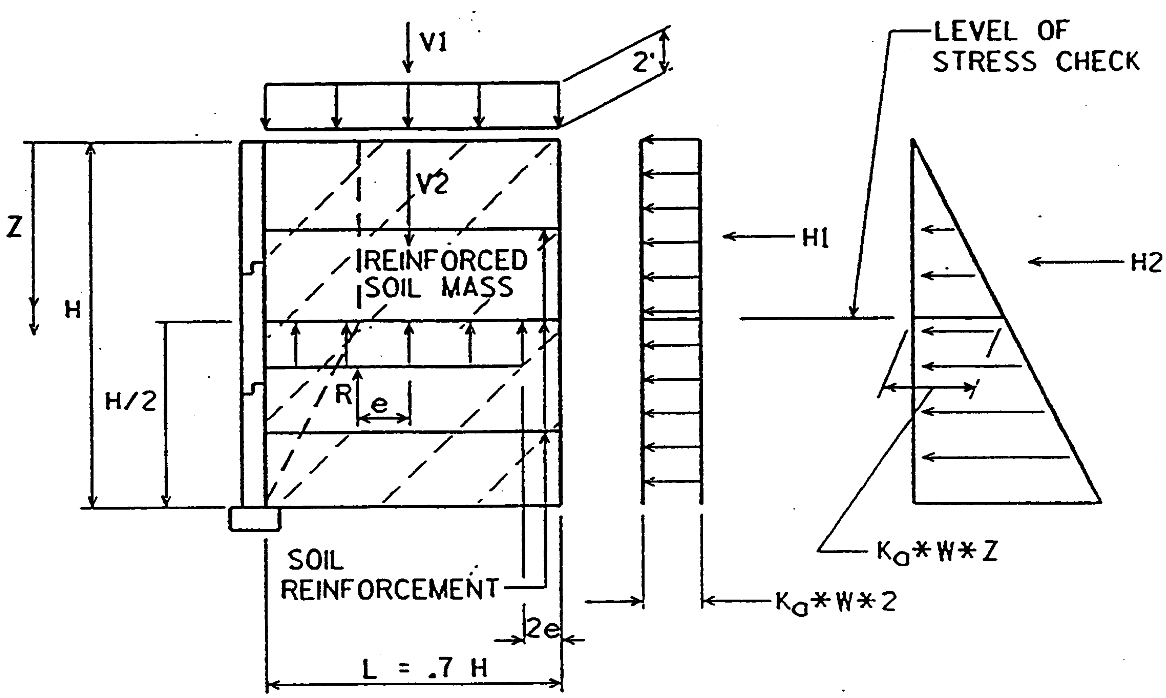


FIGURE 14.17

L = length of soil reinf. (ft)
W_r = (unit weight of soil) for retained soil (kips/cubic foot)
W_i = .12 kips/cubic foot(unit weight of soil) for reinf. infill soil
V1 = $2*W_i*L$
V2 = W_i*Z*L
H1 = Ka*0.12*2*Z
H2 = $Ka*W_r*Z*Z/2$
R = V1 + V2
Ka = see formula at beginning of design procedure
R*e= H1*Z/2 + H2*Z/3
σ_v = R/(L-(2*e))

If H = 20 & Z = 10, then Ko = (1-sinϕ_i)
Assuming ϕ_i = 34, Ko = (1.-.559) = .441 (within soil mass)
Ka = tan^2(45-17) = .283 (within soil mass)
K = .441-(.441-.283)*10/20 = .362(at Z = 10) (within soil mass)
V1 = 2*.12*14 = 3.36 kips
V2 = .12*10*14 = 16.8 kips
Assuming ϕ_r = 30°, Ka = tan^2(45-ϕ_r/2) = 0.333; and W_r = 0.120
H1 = .333*.12*2*10 = .799 kips
H2 = .333*.12*10*5 = 1.998 kips
R = V1+V2 = 20.16 kips
e = (.799*5 + 1.998*10/3)/20.16 = .528 ft.
σ_V = 20.16/(14-2.*.528) = 20.16/12.9 = 1.563 kips/ft^2
σ_H = K*σ_V = .362*1.563 = .566 kip/ft^2

Multiplying σ_H by the effective panel area to which the soil reinforcement is attached will give the force in the soil reinforcement at the failure plane.

F = .566*20/3 = 3.773 kips/ft. of wall

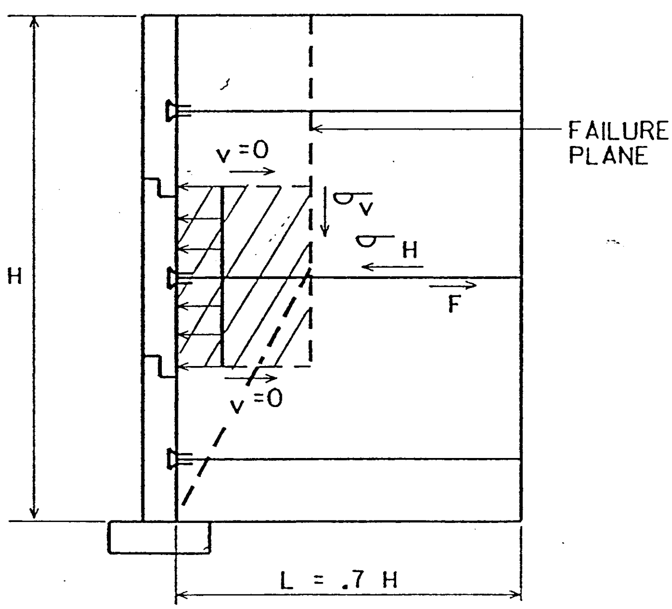


FIGURE 14.18
(Shows local equilibrium per panel. Note zero shear is assumed in soil boundaries between adjacent panels).

CASE B: Sloping Backfill (B>0°)

Find Ko, Ka, and K for infill soil at distance (z) from the top of the failure plane using Figure 14.16.

Find vertical forces V1, V2, and V3 and horizontal forces H1 and H2 using calculations accompanying external stability check for CASE B, except note that V3, H1 and H2 are based on soil above plane of reinforcement.

Then follow the procedure just outlined for CASE A to find the force in the soil reinforcement.

CASE C: Broken Back Backfill (B>0°).

Find Ko, Ka and K for infill soil at distance (z) from the top of the failure plane using Figure 14.16.

Find vertical forces V1, V2 and V3 and horizontal forces H1 and H2 using calculations accompanying external stability check for CASE C, except note that V3, H1 and H2 are based on soil above the plane of reinforcement.

Then follow the procedure just outlined for CASE A to find the force in the soil reinforcement.

Wall Anchor Force

| The force at the wall anchors (soil reinforcement attachment to wall) is assumed equal
| the force in the reinforcement at the failure plane.

The allowable force in steel reinforcement is equal to the smaller of the allowable stress times the reinforcement net area or the allowable anchor design load as described in the following paragraph. The net area of steel reinforcement shall consider deductions for bolt holes and corrosion as stated in AASHTO 5.8.6.1 during the design life (minimum 75 years) of the structure. The allowable stress for strips is .55 times the yield stress and the allowable stress for mesh/grid reinforcement shall be equal to 0.48 times the yield
| stress.

Anchor capacity must be determined from pull out tests to failure. If failure occurs after the point of attachment between reinforcement and anchor has moved more than 1/2 inch, measured from the face of the panel, then the failure load shall be that load at the time of 1/2 inch deformation. The measured failure load of the anchors shall be reduced if a loss of section of the exposed anchor components due to corrosion would result in a lower test capacity. The mode of failure shall determine if this additional reduction is necessary. The allowable design load of the anchor shall be the failure load divided by 1.5.

Allowable Anchor Design Load = Failure Load/1.5

Certified tests of anchor pullout capacity must be submitted to WisDOT for system preapproval.

Soil reinforcement must be extended beyond the failure plane a sufficient distance to develop a force equal to 1.5*F (F is the design force in the soil reinforcement) or at least 3 feet. The minimum length of the soil reinforcement is .7H, 6 feet, or (.3H_1 + 3 ft.), whichever is greater. The formulas from AASHTO 5.8.5 shall be used as follows:

For ribbed steel reinforcing strips the apparent coefficient of friction (f*) shall be 1.5 at ground level and decrease linearly to a value equal to tan ϕ_i at a depth of 20 feet, where ϕ_i is the angle of internal friction of infill. For smooth steel reinforcing strips the apparent coefficient of friction (f*) shall be 0.4 and shall be constant at all depths.

For grid reinforcing systems with transverse bar openings of, or greater than 6 inches, equation 5.8.5.-3 in AASHTO shall be used to calculate the ultimate pullout capacity.

For grid reinforcements with transverse bar spacing less than 6 inches, the ultimate pullout capacity shall be determined from equation 5.8.5-4 in AASHTO. The value of fd (coefficient of resistance to direct sliding) can be assumed to vary linearly from .45 for continuous sheets to .6 for bar mats.

Pullout resistance factors, f*, Np and fd, used to calculate the ultimate pullout capacity of steel reinforcing shall be verified by laboratory or field pullout tests. The tests shall be performed using vertical stress variations (σ_v) and reinforcement element configurations which simulate actual project conditions. The soil used in the test shall be representative of the soil meeting the material specifications for the reinforced infill. The test need only be performed once for each specific type and size of reinforcement.

Laboratory and/or field pullout tests shall be performed on samples with a minimum embedded length of 3 feet and a width representative of that used in the finished product. The pullout force shall be applied at a constant strain rate of .05 inches per minute. The ultimate pullout shall be defined as the force causing a deformation of 1/2 inch measured at the face of the confined soil block or the load causing a sudden failure, whichever is less.

The precast concrete facing panels shall be a minimum of 5.5 inches thick and contain uncoated reinforcing steel. Minimum concrete cover shall be 1 1/2 inches. The minimum area of steel shall be based on shrinkage and temperature requirements as stated in AASHTO 8.20. The panels shall be designed for the internal stresses or forces resulting from the lateral earth pressure acting on the panel and the reactions from the soil reinforcement anchors or connections. Soil reinforcement anchors shall not contact the panel reinforcing steel because of the possibility of a corrosion producing circuit developing.

Backfill for Reinforced Soil Zone

The material for infill in the reinforced soil zone shall be Grade 1 Granular Backfill as stated in 209.2 of the Wisconsin Specifications for Road and Bridge Construction except that 100% of the material shall pass the 3 inch sieve. An angle of internal friction of 34 degrees can be assumed for this material without testing.

If it is desired to use an angle of internal friction greater than 34 degrees it shall be determined by the standard Direct Shear Test, AASHTO T-236, on the portion finer than the No. 10 sieve, utilizing a sample of the material compacted to 95 percent of AASHTO T-99, Methods C or D (with oversized correction as outlined in Note 2) at optimum moisture content. No testing is required for backfills where 80% of sizes are greater than 3/4 inch.

The plasticity Index shall not exceed 6. The backfill material shall be free from organic and other deleterious materials and free of shale or other soft poor durability particles. It shall not contain foundry sand, bottom ash, blast furnace slag or other potentially corrosive material. In addition, it shall meet the following electrochemical criteria:

Resistivity	Greater Than 3000 ohm cm
pH	4.5-10
Chlorides	Less Than 100 PPM
Sulfates	Less Than 200 PPM

(2) Summary of Design Safety Factors and Requirements

a. Safety factors

Overturning	≥ 2.0
Sliding	≥1.5
Pullout	≥ 1.5
Connection Strength	≥ 1.5 @ 0.5" deformation
Global	≥ 1.3

b. Concrete Panel Facings

f'c = 4000 psi (wet cast concrete)
Min. thickness = 5.5 inches
Min. reinforcement = 1/8 square inch per foot in each direction (uncoated)
Min. concrete cover = 1.5 inches
fy = 60,000 psi

c. Traffic Surcharge

Traffic live load surcharge = 2 feet = 240 lb/ft^2

d. Retained Soil

Unit weight = 120 lb/ft^3
Angle of internal friction as determined from tests from WisDOT.

e. Design Life

75 year minimum

f. Soil Pressure Theory

| Coulomb's Theory.

g. Soil Reinforcement

Soil reinforcement may be either steel ribbed strips or steel grid systems. The minimum soil reinforcement length shall be 70 percent of the wall height and not less than 6 feet. The length of soil reinforcement shall be equal from top to bottom. Soil reinforcement must extend a minimum of 3 feet beyond the failure plane.

14.3(H) MECHANICALLY STABILIZED EARTH RETAINING WALLS WITH MODULAR BLOCK FACINGS

Concrete modular block retaining walls are proprietary wall systems. They were first introduced in 1985 and are relatively new to WisDOT. Current policy is to limit the maximum height to 22 feet (measured from the top of the leveling pad to the top of the wall).

Concrete modular block retaining walls are constructed from blocks typically weighing from 40 to 100 pounds each although blocks over 200 pounds are available. Figure 14.19 shows various types of blocks available. They are available in a large variety of facial textures and colors providing a variety of aesthetic appearances. The shape of the blocks usually allow the walls to be built along a curve, either concave or convex. The blocks or units are dry stacked meaning mortar or grout is not used to bond the units together except for the top two layers. Each proprietary system has its own unique method of locking the units together to resist the horizontal shear forces that develop. Fiberglass pins, stainless steel pins, glass filled nylon clips and mechanical interlocking surfaces are some of the methods utilized. Any pins or hardware must be manufactured from corrosion resistant materials. Some systems use blocks with voids which make the units lighter and easier to handle. During construction of these systems the voids are filled with granular material such as crushed stone or gravel. Most of the systems have a built in or automatic set-back (incline angle of face to the vertical) which is different for each proprietary system. Blocks used on WisDOT projects must be of one piece construction. A minimum weight per block or depth of block (distance measured perpendicular to wall face) is not specified on WisDOT projects. The minimum thickness allowed of the front face is 4 inches (measured perpendicular from the front face to inside voids greater than 4 square inches). Also the minimum allowed thickness of any other portions of the block (interior walls or exterior tabs, etc.) is 2 inches.

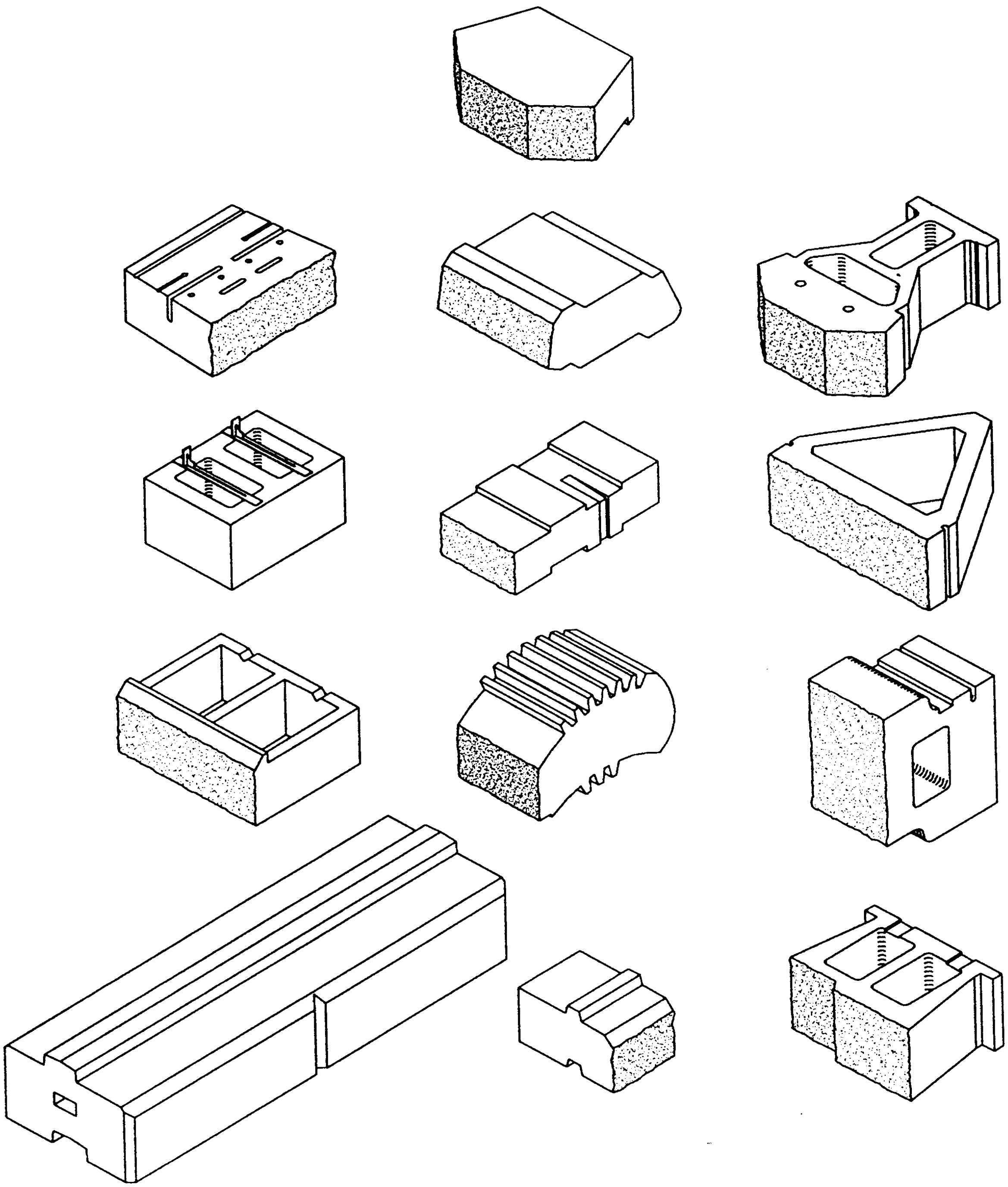


FIGURE 14.19 VARIOUS TYPES OF MODULAR BLOCKS

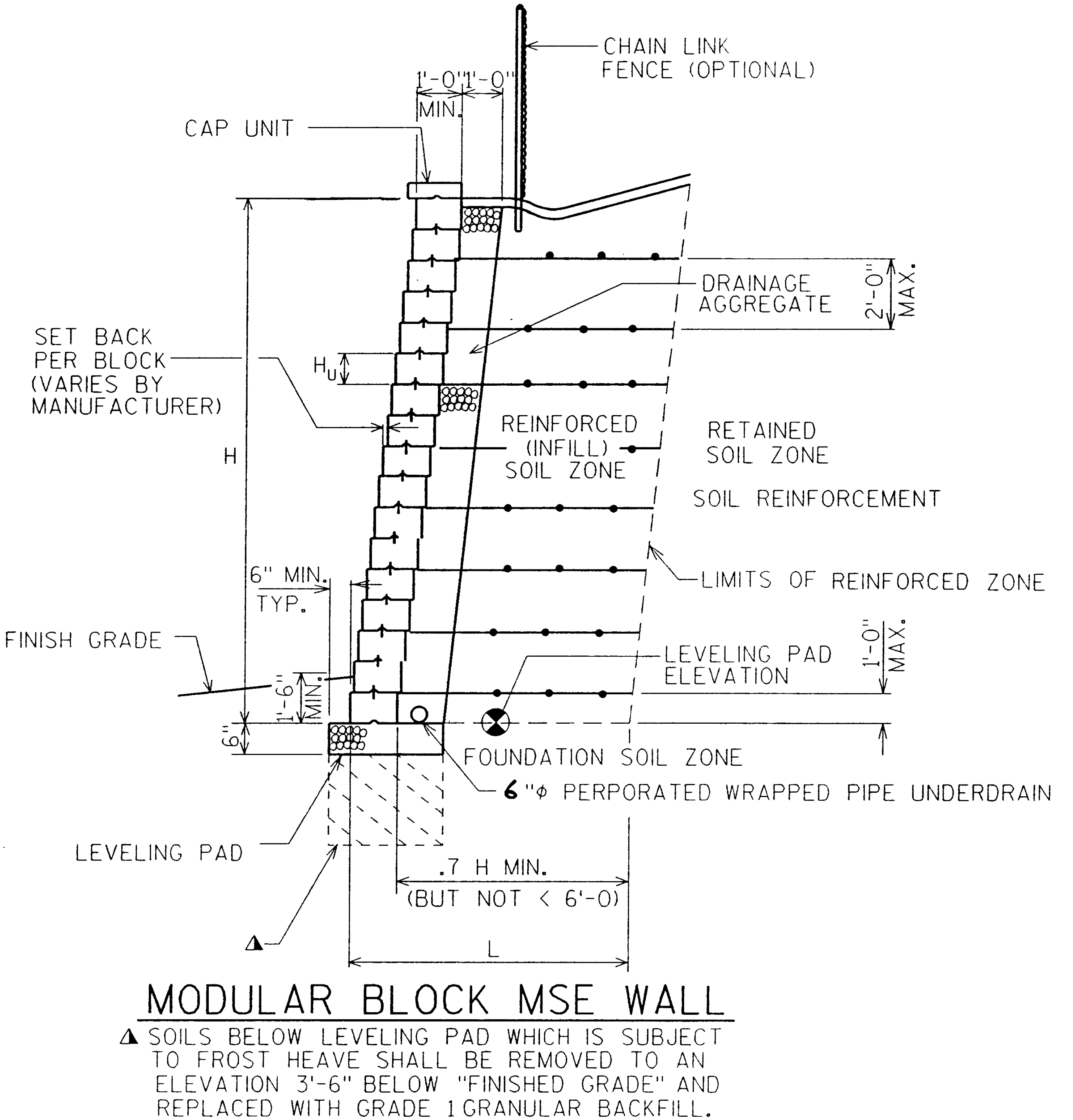

CHAIN LINK
FENCE (OPTIONAL)
1'-0" 1'-0"
MIN.
CAP UNIT
DRAINAGE
AGGREGATE
2'-0" MAX.
SET BACK
PER BLOCK
(VARIES BY
MANUFACTURER)
H_U
REINFORCED
(INFILL)
SOIL ZONE
RETAINED
SOIL ZONE
H
SOIL REINFORCEMENT
LIMITS OF REINFORCED ZONE
6" MIN.
TYP.
FINISH GRADE
1'-6" MIN.
LEVELING PAD
ELEVATION
1'-0" MAX.
6"
FOUNDATION SOIL ZONE
6"φ PERPORATED WRAPPED PIPE UNDERDRAIN
LEVELING PAD
.7 H MIN.
(BUT NOT < 6'-0)
L
MODULAR BLOCK MSE WALL
SOILS BELOW LEVELING PAD WHICH IS SUBJECT
TO FROST HEAVE SHALL BE REMOVED TO AN
ELEVATION 3'-6" BELOW "FINISHED GRADE" AND
REPLACED WITH GRADE 1 GRANULAR BACKFILL.

FIGURE 14.20

Modular block walls are founded on six inch unreinforced concrete leveling pad foundations. If differential settlement occurs a block may crack due to bending from beam action with supporting reactions developing at its ends rather than the typical uniform reaction along its entire bottom area. Edge pressure may also cause spalling of the block. Differential settlement may also reduce the design pullout capacity of the soil reinforcement from the blocks since pullout capacity tests are based on the blocks being in full contact. Differential settlement is limited to 1/200 for MSE modular block walls without full height separation joints. When such joints are used, the differential settlement in wall sections between joints is limited to 1/200. Separation joints should also be included at any point of anticipated abrupt change in vertical movement.

Soil reinforcement for modular block MSE walls may be either geosynthetics (extensible) or metallic (inextensible).

Geosynthetics shall be geogrids. Geogrids, which are manufactured from long chain polymers, may be either polyester (PET), polypropylene (PP), or high density polyethylene (HDPE). Because of degradation by hydrolysis in an alkaline environment the use of polyester (PET) is not allowed if the reinforced soil zone is subject to exposure from deicing salts or fertilizer. This restriction does not apply if an impervious membrane is installed at the top of the reinforced soil zone.

The maximum vertical spacing allowed between layers of adjacent soil reinforcement shall not exceed 2 times the block depth (front face to back face) or 32 inches, whichever is less. The first (bottom) layer of reinforcement shall be placed no further than 12 inches (average) above the top of the leveling pad but at least one block height above the leveling pad. The last (top) layer of soil reinforcement shall be no further than 24 inches (average) below the top of the block finished grade.

The ASTM tests required on the geogrids to determine their design parameters are stated under "Design Procedures for Modular Block MSE Walls". Certified pullout tests for the combination of modular block and soil reinforcement are required as part of the preapproval process. The results of the specific pullout test used for the design shall be included in the design drawings and calculations submitted to WisDOT for preapproval.

The pullout resistance between geogrids and the modular blocks is developed from any one or a combination of the following:

a. Friction between the concrete and geogrid.
b. Interlock with lips or keys in block.
c. Friction and/or interlock with crushed stone infill of hollow core blocks.
d. Pins or clips between blocks passing thru geogrids.

The polymeric material (plastics) used for geogrids typically exhibit short-term strengths 3 to 4 times greater than their long-term strengths (75 years or more). Moduli of elasticity will also decrease with time. The reduction of these mechanical properties of plastics are time, stress, and temperature dependent and the purpose of the required testing is to determine what the mechanical properties will be at the end of the design service life (minimum 75 years). The 75 year properties are then used for design. The required tests are stated under "Design Procedures for Modular Block MSE Walls" The wall supplier is responsible for procuring and delivering the geogrid to the job site and assumes all responsibility for geogrid being in compliance with specifications.

The requirements for metallic soil reinforcement are identical to those stated in Section 14.3(G) of this manual.

(1) Design Procedure for Modular Block MSE Walls

The Design Procedure for 2 Cases are presented:

CASE A -- Horizontal Backslope (B=0°) & Sloping Backfill (B>0°)
CASE B -- Broken Back Backfill (B>0°)

For walls with zero setback and level backfill the active earth pressure coefficient for external stability shall be determined from the following formula.

$$Ka = COS\,B * (COS\,B - X) / (COS\,B + X)$$

$$X = SQRT\,(COS^2 B - COS^2 \phi_r)$$

For walls with setbacks one shall use Ka from the following formula (Coulombs Formula), with δ equal to zero.

$$Ka = \frac{\cos^2(\phi_r + A)}{\cos^2 A \cos(A-\delta)\ \left(1 + (Z/Y)^{1/2}\right)^2}$$

$Z = \sin(\phi_r + d)\sin(\phi_r - B)$

$Y = \cos(A-\delta)\cos(A+B)$

ϕ_r = angle of internal friction of the retained soil
B = backslope angle (see Figure 14.21)
A = wall setback angle from vertical
δ = interface friction angle between the reinforced soil zone and the retained soil. (use δ = 0.)

Use the angle of internal friction (ϕ_r) supplied by WisDOT.

CASE A: SLOPING OR HORIZONTAL BACKFILL (B≥0°)

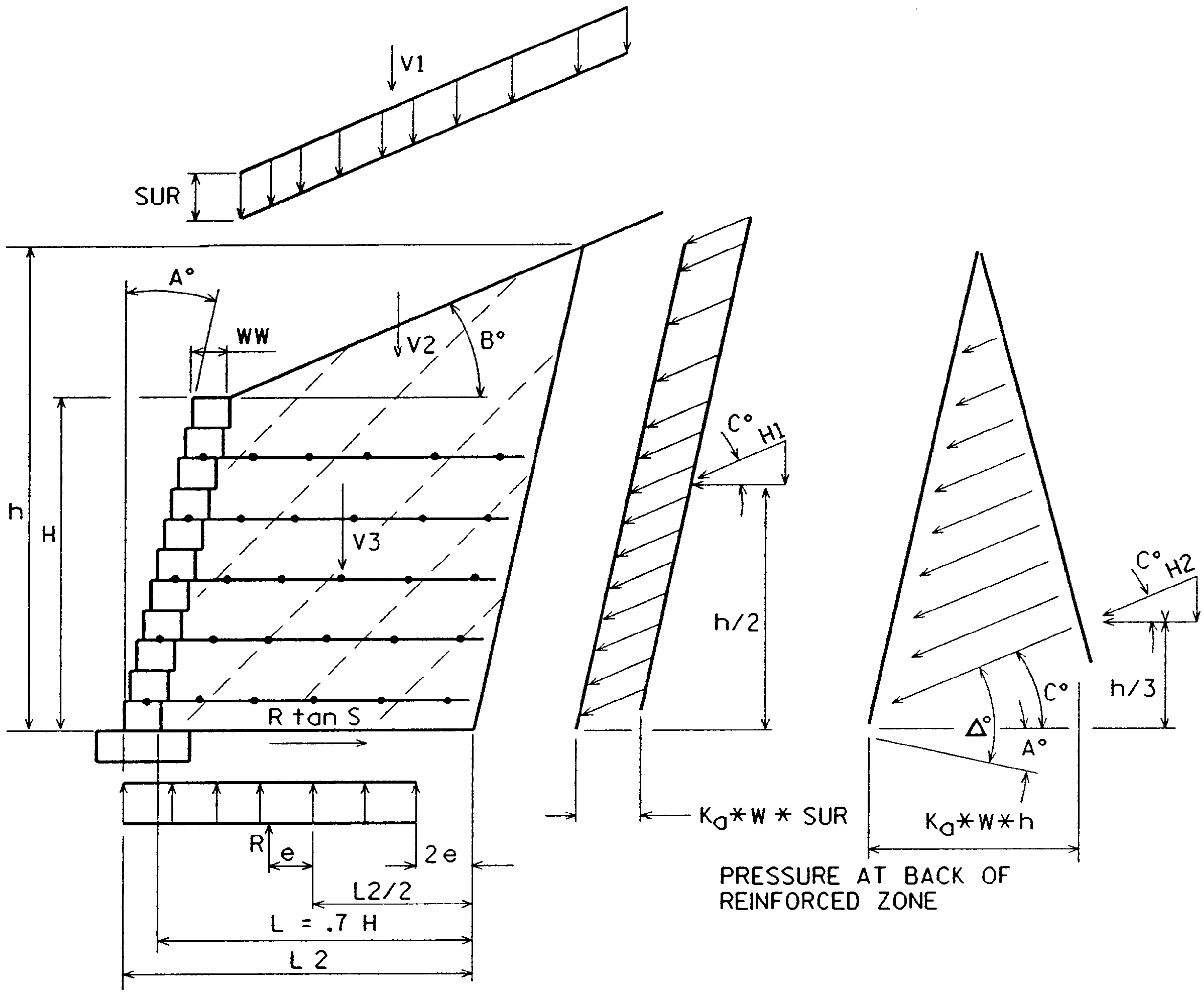


FIGURE 14.21
EXTERNAL STABILITY CALCULATIONS

Delta = external interface friction angle
Delta = the lesser of ϕ_i or ϕ_r where

ϕ_i = angle of internal friction of reinforced infill soil.
ϕ_r = angle of internal friction of retained soil.

C° = Delta - A° Note: C cannot exceed angle B.
h = H + tanB (L+L*tanB*tanA)
W_i = .12 kips/cubic foot (unit weight of soil) for reinf. infill soil
V1 = SUR*W_i(L+(h-H)*tanA)
V2 = W_i*(h-H)*L/2.
V3 = W_i*H*L2
| H1 = SUR*Ka*0.12*h
H2 = Ka*W_r*h*h/2.
R = V1+V2+V3+sinC(H1+H2)

<u>Factor of safety against overturning =</u> ∑ Resisting Moments/∑ Overturning Moments ≥ 2.0

V3*(L2+H*tan A)/2 + V2*(H*tanA+WW+2/3L)
+H1*sinC*(L2+h/2*tanA)+H2*sinC(L2+h/3tanA)/[H1*cos C*h/2 + H2*cos C*h/3] ≥ 2.

Note: For overturning and sliding analysis, surcharge is assumed to act outside of the reinforced soil zone and therefore V1 is not used. V1 is used to compute maximum bearing pressure.

<u>Factor of safety against sliding friction =</u> ∑ Resisting Forces/∑ Driving Forces ≥1.5

(V2+V3)tanS/cosC(H1+H2) ≥ 1.5

where tanS is the coefficient of sliding friction (tanS to be provided by WisDOT).

Also sliding should be checked at the level of first geogrid from the bottom using the geogrid coefficient of direct sliding but including the shear strength between modular block units. If the geogrid coefficient of direct sliding is unknown use .65 of tanS.

The bearing pressure at the bottom of the reinforced soil mass and blocks, BP, is determined by using the Meyerhof stress distribution.

BP = R/(L2-(2*e))
e is determined by taking moments about the center of base length L2.
e = [H1*cos C*h/2+H2*cos C*h/3-H1*sin C(L2/2+h/2*tan A)-H2*sin C (L2/2+h/3*tanA)-V1[(h+H)/2*tanA+WW/2]-V2*(H*tanA+2L/3+WW-L2/2)-V3*(H/2*tan A)]/R
BP ≤ "allowable bearing pressure"

The "allowable bearing capacity" is provided by the WisDOT and must be greater than or equal to "BP".

CASE B: BROKEN BACK BACKFILL (B>0°)

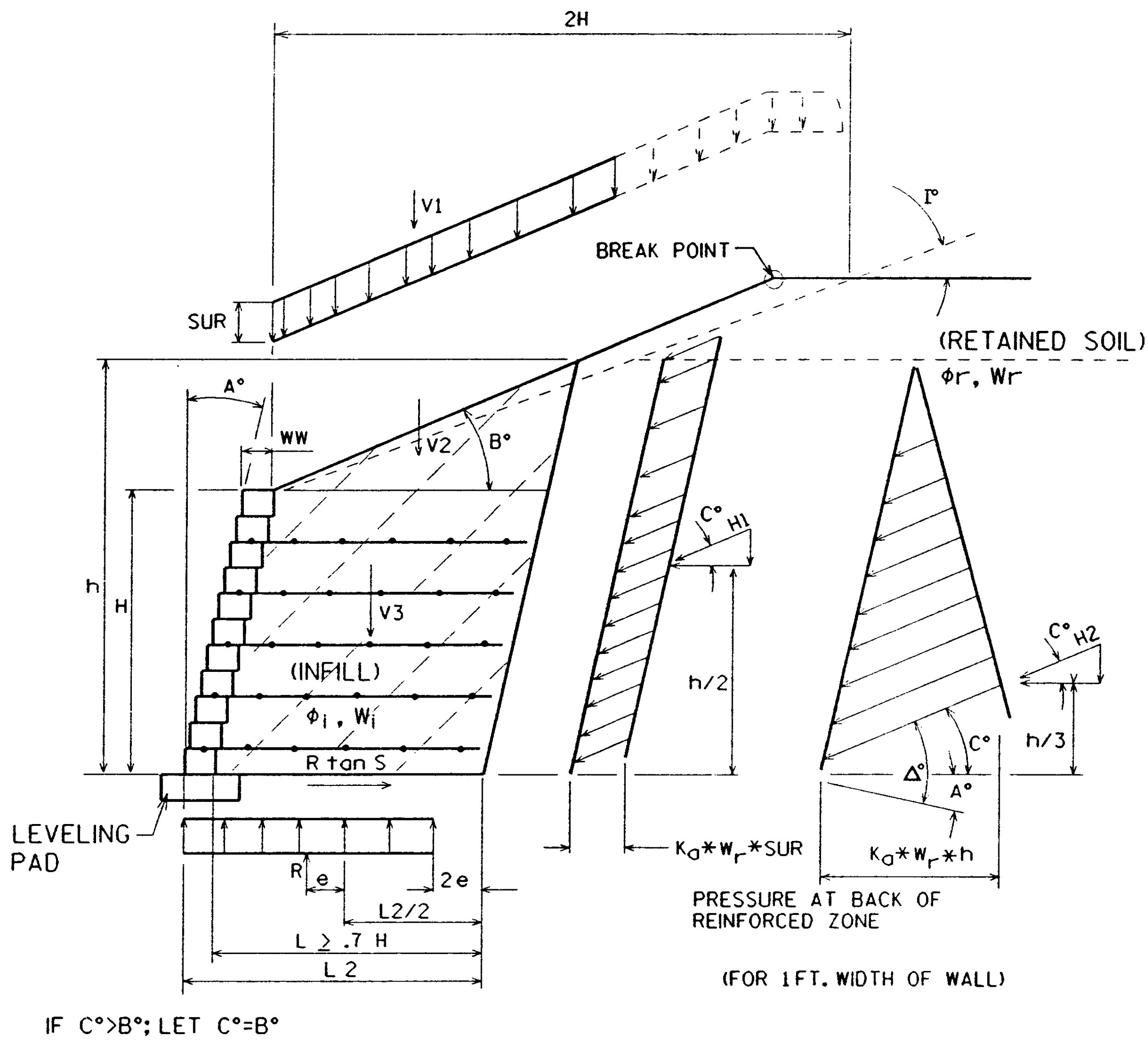


FIGURE 14.22
EXTERNAL STABILITY CALCULATIONS

Delta = the smaller of ϕ_i and ϕ_r

ϕ_i = angle of internal friction of reinforced infill soil

ϕ_r = angle of internal friction of retained soil

If a break in the slope behind the wall is located horizontally within two times the height of the wall 2H, a broken back slope design CASE B may be used. If the break is located at 2H or more from the wall, CASE A is to be used.

The only difference between CASE A and CASE B is the magnitude for forces H1 and H2. The magnitude of these forces is a function of the active earth pressure coefficient Ka which is shown at beginning of design procedure. CASE A and CASE B both use this formula for Ka, but for CASE B substitute I° for B° in the formula. When the break in the slope behind the wall is located horizontally within 0.5H from the backface of the reinforced soil mass, where H is the height of the wall, then liveload surcharge is considered as part of the design. If the break is located at 0.5H or more from the backface of the reinforced soil mass, liveload surcharge is not considered as part of the design.

Note: C° cannot exceed angle B°.

C° = Delta -A°
h = H+tanB°(L+L*tanB°*tanA°)-(ft.)
L = length of soil reinf. (ft.)
W_r = (unit weight of soil) for retained soil (kips/cubic foot)
W_i = 0.12 kips/cubic foot (unit weight of soil) for reinf. infill soil
V1 = Sur*W_i(L+(h-H)*tanA) (kips)
V2 = W_i (h-H)*L/2 (kips)
V3 = W_i*H*L2 (kips)
H1 = Ka*0.12*Sur*h (kips)
H2 = Ka*W_r*h*h/2 (kips)
R = V1 + V2 + V3 +sinC°(H1+H2) (kips)
Ka = see formula at beginning of design procedure. (But replace B° with I°)
tanS = coefficient of friction

Analysis of Overturning - AASHTO 5.5.5

Factor of safety against overturning = ∑ Resisting Moments/∑ Overturning Moments ≥ 2.0

V3*(L2+HtanA)/2 + V2*(HtanA+WW+2/3L)
+H1sinC°(L2+h/2*tanA)+H2sinC°(L2+h/3*tanA)/
[H1*cosC*h/2+H2*cosC°*h/3] ³ 2.0

Note: For overturning and sliding analysis, surcharge is assumed to act outside of the reinforced soil zone and therefore V1 is not used. V1 is used to compute maximum bearing pressure.

Analysis of Sliding - AASHTO 5.5.5

Factor of safety against sliding = ∑ Resisting Forces/∑ Driving Forces ≥ 1.5.

$$(V2+V3)\tan S/\cos C°(H1+H2) \geq 1.5$$

where tanS is coefficient of sliding friction (tanS to be provided by WisDOT).

Sliding should also be checked at the level of first geogrid from the bottom using the geogrid coefficient of direct sliding but including the shear strength between modular block units. If the geogrid coefficient of direct sliding is unknown use .65 of tanS.

<u>Analysis of Soil Bearing Pressure</u> - AASHTO 5.8.3

The bearing pressure at the bottom of the reinforced soil mass and blocks, BP, is determined by using the Meyerhoff stress distribution.

$$BP = R/(L2-(2*e))$$

e is determined by taking moments about the center of base length L2.

$$e = \left[H1\cos C°*h/2+H2*\cos C°*h/3-H1*\sin C°\left(\frac{L2}{2} + \frac{h}{2}*\tan A\right)-H2*\sin C°*\left(\frac{L2}{2}+h/3*\tan A\right)-V1\left(\frac{h+H}{2}*\tan A+\frac{WW}{2}\right)-V2\left(H*\tan A+\frac{2L}{3}+WW-\frac{L2}{2}\right)-V3(H/2*\tan A)\right]/R$$

BP ≤ "allowable bearing capacity"

The previous equations for "h, V2, Factor of Safety against overturning and Bearing Pressure" in CASE B are correct when the break point of the slope is greater than or equal to "L" from the backface of the wall as shown in Figure 14.22. If the break point is less than "L", away, then these equations will have to be modified.

The "failure plane" for modular block MSE walls reinforced with geogrid reinforcement (extensible) is defined by a straight line passing through the heel (retained earth side) of the lower most block at an angle α from the horizontal. See Section 14.3(G) for the failure plane for modular block MSE walls reinforced with metallic (inextensible) reinforcement. σ is calculated from the following equation.

$$\tan(\alpha-\phi) = \frac{-X + (X*(X+Y)(1+Y*\tan(\delta-A)))^{1/2}}{1+\tan(\delta-A)(X+Y)}$$

where $X = \tan(\phi_i-B)$

$Y = \cot(\phi_i+A)$

ϕ_i = angle of internal friction of reinforced infill soil

δ = angle of friction at back of wall (assume = 2/3 ϕ_i)

See Figure 14.23 & 14.24 for definition of A, B & a.

The failure plane for CASE A and B with extensible reinforcement is based on angle (α) as calculated above.

The horizontal stress σ_h at each reinforcement level for extensible reinforcement may be computed by multiplying the vertical stress σ_v at that level by the active earth pressure coefficient Ka.

$$Ka = \frac{\cos^2(\phi_i + A)}{\cos^2 A \cos(A-\delta)\,(1 + (Z/Y)^{1/2})^2}$$

$$Z = \sin(\phi_i + \delta)\sin(\phi_i - B)$$

$$Y = \cos(A-d)\cos(A+B)$$

| where ϕ_i is the peak angle of internal friction of the reinforced soil zone (30 degrees) δ is the interface friction angle which is assumed to be 2/3 times the
| angle of internal friction of the reinforced infill soil. (20 degrees).

For steel (inextensible) reinforcement use the value of (K) described in Figure 14.16 in place of Ka shown above.

The vertical stress σ_v is based on the vertical loads being distributed over a length determined by the Meyerhoff formula. Apply the same procedure that is used to calculate the maximum bearing pressure at the bottom of the reinforced soil mass shown in the external stability equations. The same equations can be used except "h" and "H" must be decreased by the distance from the top of the leveling pad to the level of geosynthetic reinforcement where vertical earth pressures are being calculated. Should this procedure result in R falling to the right of center of L2 (see Figure 14.21) then calculate σ_v based on the height of overburden plus surcharge at the center of the contributing area (La) for the geosynthetic reinforcement being considered. (d and La value in Figure 14.24).

<u>Soil Reinforcement Forces</u> (Extensible and Inextensible)

AASHTO 5.8.4.1 says to include surcharge for stress calculations. The force in the soil reinforcement is determined at the location of the failure plane as follows:

CASE A: HORIZONTAL BACKSLOPE (B=0°)

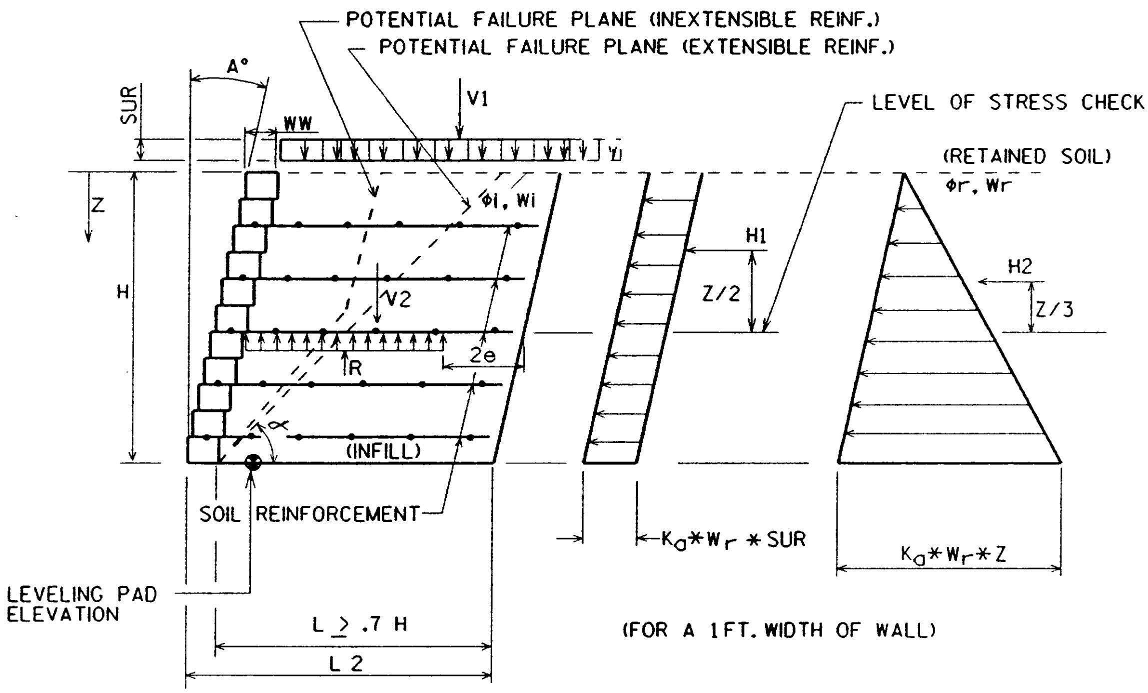


FIGURE 14.23

Delta = the smaller of ϕ_i and ϕ_r

ϕ_i = angle of internal friction of reinforced infill soil

ϕ_r = angle of internal friction of retained soil

C° = Delta -A° Note: C° cannot exceed angle B°
L = length of soil reinforcement (ft.)
W_r = (unit weight of soil) for retained soil. (kips/cubic foot).
W_i = 0.12 kips/cubic foot (unit weight of soil) for reinf. infill soil.
V1 = sur*W_i*L (kips)
V2 = W_i*Z*L (kips)
H1 = Ka*0.12*Sur*Z (kips)
H2 = Ka*W_r*Z*Z/2 (kips)
R = V1+V2+sinC°(H1+H2) (kips)
Ka = see formula at beginning of design procedure

$$R^*e = [H1\cos C^\circ {}^* Z/2 + H2\cos C^\circ {}^* Z/3 - H1\sin C^\circ\left(\frac{L2}{2} + (H-Z+Z/2)^*\tan A\right)$$
$$- H2\sin C^\circ\left(\frac{L2}{2} + (H-Z+Z/3)^*\tan A\right) - V1\left(\frac{L2}{2} + H\tan A - \frac{L}{2}\right)$$
$$- V2((H-Z+Z/2)^*\tan A)$$

σ_v = R/(L-(2*e))

If alternate method is required to calculate σ_v as described previously.

σ_v = (d+sur)*W_i

where d and its location are shown in Figure 14.24.

Then for extensible reinforcement: σ_H = Ka*σ_v; where Ka is based on formula found on previous pages. For inextensible reinforcement: σ_H = K*σ_v; where K is defined on Figure 14.16.

Multiplying σ_H times its contributing area will provide the force in the soil reinforcement (Fg). This is the force in the reinforcement at the failure plane. Because geogrid reinforcement is continuous the contributing area is the vertical spacing and the resulting force is on a per foot basis.

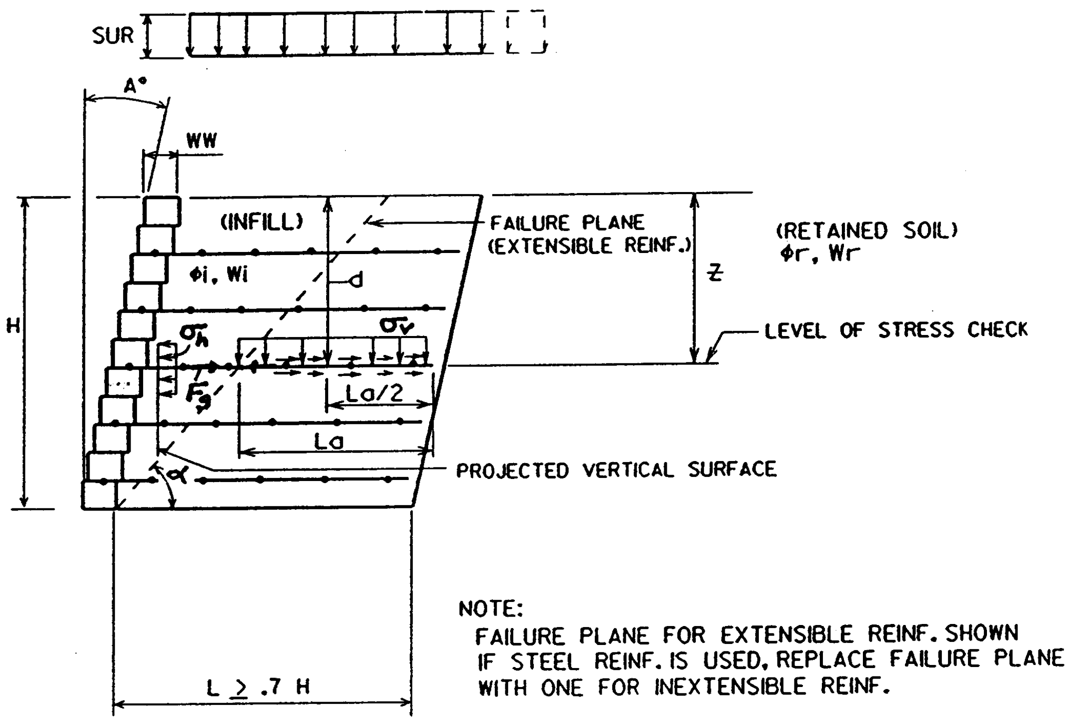


FIGURE 14.24

Force at Block

The force in the geogrid reinforcement at the back face of the block and at the failure plane are assumed to be equal.

The force "F" in the geogrid is equal to s_h times the contributory area. Since geogrid reinforcement is continuous the contributing height is normally used.

CASE A: Sloping Backfill (B>0°)

Find vertical forces V1, V2 and V3 and horizontal forces H1 and H2 using calculations accompanying external stability check for CASE A, except note that V3, H1 and H2 are based on soil above plane of reinforcement. Then follow the procedure just outlined for CASE A (Horizontal Backslope) to find the force in the soil reinforcement.

CASE B: Broken Back Backfill (B>0°)

Find vertical forces V1, V2 and V3 and horizontal forces H1 and H2 using calculations accompanying external stability check for CASE B, except note that V3, H1 and H2 are based on soil above the plane of reinforcement. Then follow the procedure just outlined for CASE A (Horizontal Backslope) to find the force in the soil reinforcement.

The connection strength between a geogrid and the blocks must be determined by National Concrete Masonry Association (NCMA) Test Method SRWU-1. The service state connection strength shall be based on a deformation of the geogrid relative to the block, measured at the face of the blocks of .75 inches (19 mm). The connection strength used for design shall be the lessor of the "Peak Connection Strength" and the "Service State Connection Strength". The "Connection Strength" divided by the force in the geogrid ("F") must equal or exceed 1.5.

"Connection Strength"/F $\geq$ 1.5

The allowable force ("F") in the geogrid reinforcement shall be in accordance with AASHTO 5.8.7.2. The 10,000 hour creep test shall be performed in accordance with ASTM D5262, Tension Creep Testing of Geosynthetics, or GRI Test Method, GG-3(a & b), Tension Creep Testing of (Stiff and Flexible) Geogrids. One suggested way to extrapolate the performance of geogrids beyond the 10,000 hour range to the design life of the wall, procedures outlined in ASTM D2837-90, Obtaining Hydrostatic Design Basis for Thermoplastic Pipe Materials and ASTM D1598-91, Time-to-Failure of Plastic Pipe Under Constant Internal Pressure shall be utilized. From these tests the values of T_1 (Limit State Tensile Load) and T_W (Serviceability State Tensile Load) as described in AASHTO 5.8.7.2 shall be determined.

The values of FD (factor of safety or reduction factor for polymeric reinforcements with respect to environmental and aging losses) and FC (factor of safety or reduction factor with respect to construction damage) shall be determined from tests. In the absence of job specific tests, FD of 2.0 shall be used. When test data is available the minimum value of FD shall be 1.1.

When no test data for FC is available a value of 3 shall be used. When test data is available the minimum value of FC shall be 1.3.

In addition to FD and FC an overall factor of safety FS equal to 1.78 is applied. (AASHTO 5.8.7.2)

Pullout Capacity of Soil Reinforcement

The pullout capacity of geogrids is developed by extending the geogrid beyond the failure plane a sufficient distance to develop a force (FU) equal to 1.5*F, where F is the force in the geogrid at the failure plane. The minimum length of the soil reinforcement is .7H, 6 ft., or the distance to the failure plane + 3 feet whichever is greater. FU shall be calculated from the following formula.

$$FU = 2 \tan \phi_i \, \sigma_v \, L_A \, fd$$

where FU = "pullout capacity" per unit width of tensile reinforcement (lbs/ft)

σ_v = vertical stress (lbs/ft^2)
(σ_v = d * 120 in Figure 14.24)
La = length of reinforcement beyond the failure plane
fd = equivalent coefficient of direct sliding derived from pullout tests.
ϕ_i = angle of internal friction of reinforced soil zone. (Use 34°)
For design, the force in the soil reinforcement (f) cannot exceed 2/3 of FU, ie, FU/F $\geq$ 1.5.

For geogrids, fd is a function of the open area of the grid. In the absence of product specific data tested with site specific granular backfills it may be estimated from the following table:

% Open Area of Grid	fd
80% or more	0.5
51-79%	0.7
50% or less	0.6

For geogrids, grid pullout from the soil is obtained from a combination of soil shearing on plane surfaces parallel to the direction of grid movement and soil bearing on transverse grid surfaces normal to the direction of grid movement.

For geogrids the pullout resistance shall be evaluated from the results of the pullout tests. The use of direct shear tests to determine the pullout resistance of geogrids is not acceptable. Ultimate pullout capacity shall be based on a maximum elongation of the embedded geogrid of 0.5-inch as measured at the leading edge of the compressive zone within the soil mass and not the ultimate pullout capacity. The soil used in the pullout tests shall be representative of the soil meeting the material specifications for the reinforced infill.

The pullout shall be performed using vertical stress variations (σ_v) and reinforcement element configurations simulating actual project conditions.

The pullout tests shall be performed on samples with a minimum embedded length of 2.0 feet. The tests shall be performed on samples with a minimum width of 1.0 feet or a width equal to a 4 longitudinal grid element, whichever is greater. Tests shall be conducted at 70 ± 2°F at constant strain rates of 0.02 in. per minute.

(2) Summary of Design Safety Factors and Requirements

a. Safety Factors

Overturning	≥ 2.0
Sliding	≥ 1.5
Pullout	≥ 1.5
Connection Strength	≥ 1.5 @ .75" deformation
Overall Safety Factor (Limited State determination)	≥ 1.78
Global	≥ 1.3

b. Block Data

One piece block.
Minimum thickness of front face = 4 inches.
Minimum thickness of internal cavity walls other than front face = 2 inches. 28 day concrete strength = 5000 psi. Maximum water absorption rate by weight = 5%. Sealer required on walls with significant exposure to salt.

c. Traffic Surcharge

Traffic live load surcharge = 2 feet = 240 lb/ft^2.

d. Retained Soil

Unit weight = 120 lb/ft^3
Angle of internal friction as determined from tests from WisDOT.

e. Design Life

75 years minimum

f. Soil Pressure Theory

| Use Coulomb's Theory.

g. Soil Reinforcement

Soil reinforcement may be either geogrid material or a metallic reinforcement. The minimum soil reinforcement length shall be 70 percent of the wall height and not less than 6 feet. The length of soil reinforcement shall be equal from top to bottom and the maximum vertical spacing between layers of soil reinforcement shall be 2 times the block depth (front face to back face) or 32 inches, whichever is less. Soil reinforcement must extend a minimum of 3 feet beyond the failure plane.

14.3(I) PREFABRICATED MODULAR BIN AND CRIB WALLS

Concrete modular bin and crib wall systems function as gravity walls utilizing their own weight and the weight of the soil infill to resist overturning and sliding. They are proprietary wall systems whose design is provided by the wall supplier. Bins usually consist of interlocking cell like reinforced concrete structures of four walls or two walls with interconnecting diaphragms. Tops and bottoms of the cells are usually open or partially open. Cribs consist of solid interlocking reinforced concrete members called rails and tiebacks (sometimes called stretches and headers). The rails run parallel with the wall face at both the front and rear of the cribbing and the tiebacks run transverse to the rails to tie the structure together. Rails and cross sections of tiebacks form the front face of the wall. A bin system may utilize partial height walls at the front face or both faces so that the infill soil surface is exposed allowing plant growth in the infill soil. Crib walls can also be constructed with horizontal setbacks to expose the infill soil for supporting plant growth. Walls designed with exposed infill soil that will support plant growth are commonly referred to as living walls. Open faced bin wall systems can be filled with rock or stone if a living wall is not required.

The base width of a bin or crib wall ranges from .4 to .5 of the wall height for walls with no surcharge and small backslopes. For walls with surcharge and/or large backslopes the base width required could reach .6 the wall height. The minimum embedment for bin or crib walls is 1'-6" to the top of the footing or leveling pad or as given in Section 5.8.1 of AASHTO, whichever is greater.

(1) Design Procedure for Modular Bin and Crib Walls

AASHTO Specifications for Highway Bridges, Section 5.9, "Prefabricated Modular Wall Design" shall be used when appropriate for the design of modular crib and bin walls. Article 5.9.2 allows 80% of the weight of the soil in-fill material to be effective in resisting overturning moments. The basis of this practice is empirical and recognizes the fact that some of the soil in the modules is in direct contact with the foundation soil. A value greater than 80% is allowed if the actual value can be verified by full scale field tests or if the bins are constructed with floors.

Some concrete modular crib or bin systems are relatively rigid and are subject to structural damage due to differential settlements, especially in the longitudinal direction. Therefore, the ultimate bearing capacity for footing design may be comparable to those for cast-in-place walls because both are relatively sensitive to differential settlements.

AASHTO Article 5.9.4 states that the inside pressure in the bin shall be the same for each module and shall not be less than:

$P_i = wb$ where:

P_i = pressure inside bin module

b = inside distance between bin walls

w = unit weight of infill

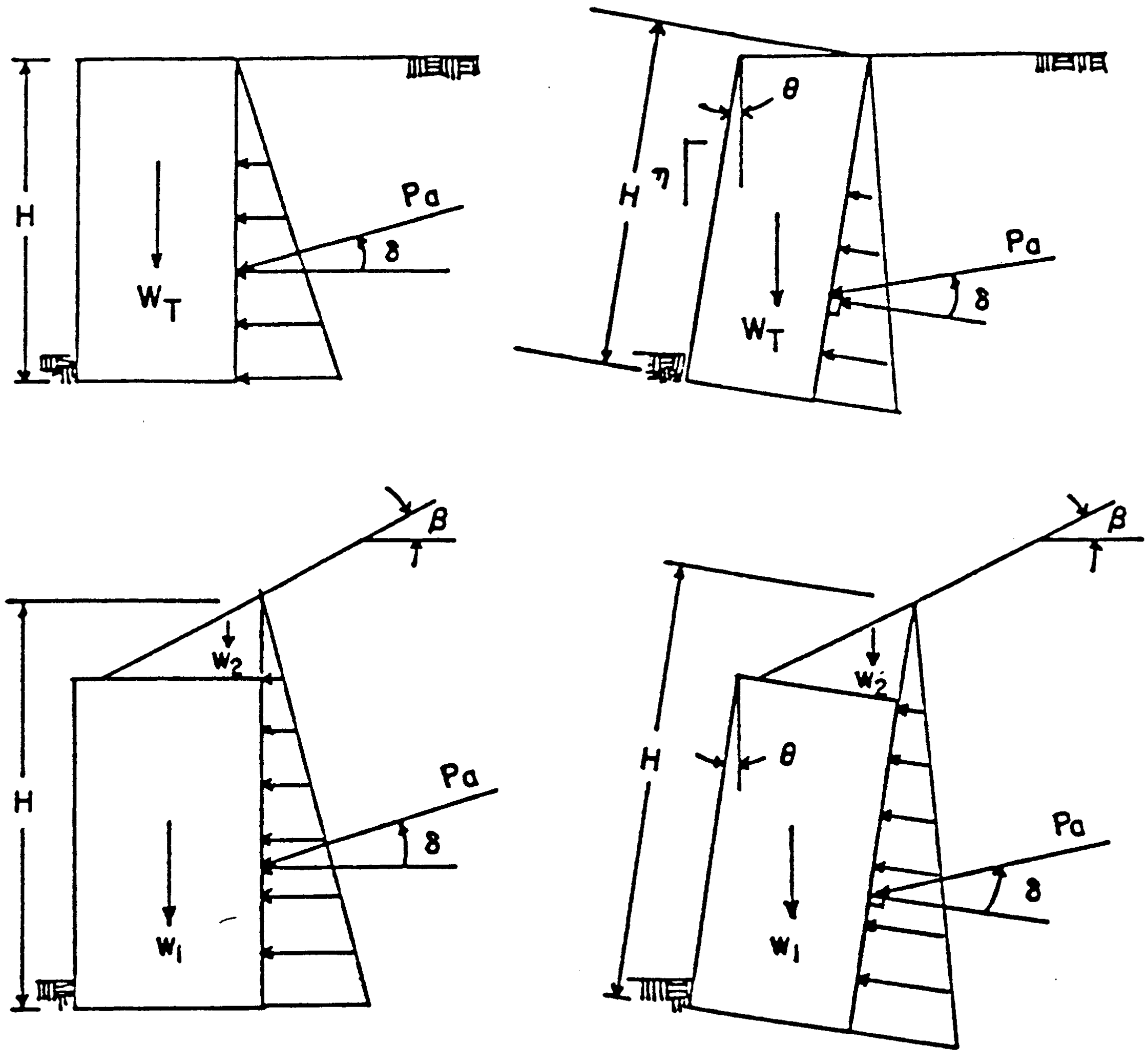
This formula comes from the theory for an infinitely long silo trench of width b. Small rectangular sections are theoretically subject to smaller bin pressures. The exact formula is:

$$P_i = w\ b/2\ \tan\delta$$

where δ is the angle of wall friction which in granular soil can be taken as equal to 30 degrees. Since the tangent of 30° = .577 the formula conservatively becomes P_i = wb which is the formula for the maximum inside bin pressure. The maximum pressure occurs at a depth equal to 3 times b and remains constant at deeper depths. Note that this formula is for vertical face bins and that bins with sloped walls will require special analysis. Rear walls of bins shall also be designed for the maximum external soil pressure acting simultaneously with 1/2 of the maximum internal wall pressure. Rails and tiebacks of modular crib systems shall be designed for the shear, bending and tension forces that develop due to dead loads, earth loads and live load surcharge. Load factor design shall be used.

Stability computations for crib and bin modular systems shall be made by assuming that the system acts as a rigid body. Lateral earth pressures can be computed by Coulomb's theory. When the rear of the concrete modules form an irregular surface (stepped surface) pressures shall be computed on an average plane surface drawn from the lower back heel of the lowest module as shown in Figure 14.26. The angle of friction (d) between the back of the modules and backfill is stated in AASHTO Article 5.9.2 for three possible cases. This angle effects the magnitude and direction of the resulting lateral earth pressure force. The method shown in Section 14.3 C, (1) "Design Procedures for Gabion Walls" may also be used for modules on level footings. For overturning, moments shall be taken about the toe of the lower unit. When performing an external analysis of the system, only the forces acting on or inside the pressure plane may be utilized. The concrete for modular bin and crib walls shall have a minimum 28 day compressive strength of 4000 psi. Reinforcement shall be uncoated Grade 60 either bars or welded wire fabric. Infill soil shall be Grade 1 Granular Backfill (209.2) for systems which have a closed front or as stated in the contract documents for systems which have an open front (living wall systems). Closed front systems require 1 foot of drainage aggregate directly behind the wall unless the front face joints are covered on the inside face with a geosynthetic to prevent soil from passing thru the joints.

Some modular systems are available with special surface finishes such as exposed aggregate and various striated patterns which greatly enhance the appearance of the walls. Concrete stains can be used to add color to normal surface finishes.



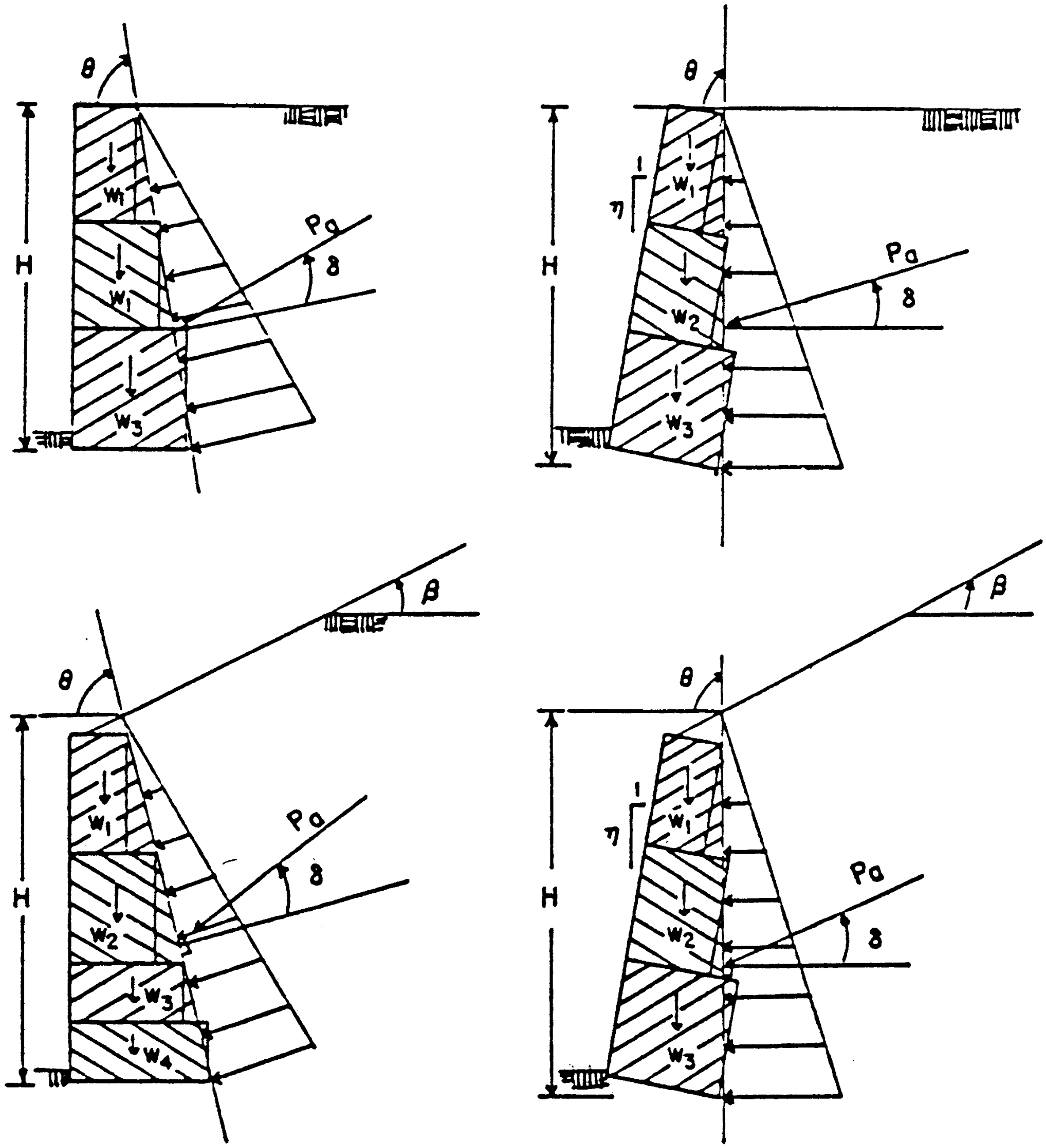
$$P_a \text{ @ } \frac{H}{3}$$

$$P_a = \frac{1}{2} \gamma H^2 K_a$$

$$K_a = \frac{\cos^2(\phi+\theta)}{\cos^2\theta \cos(\theta-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\theta-\delta)\cos(\theta+\beta)}}\right]^2}$$

FIGURE 14.25

LATERAL EARTH PRESSURE ON CONCRETE MODULAR SYSTEMS OF CONSTANT WIDTH



$$K_a = \frac{\sin^2(\theta+\phi)}{\sin^2\theta \sin(\theta-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\theta-\delta)\sin(\theta+\beta)}}\right]^2}$$

FIGURE 14.26

LATERAL EARTH PRESSURE ON CONCRETE MODULAR SYSTEMS OF VARIABLE WIDTH

(2) Analysis of Concrete Modular Bin System

Example: Uniform Bin Sizes

Given:

B = 15°
A = 20°
ϕ_r = 34° (From Tests)
W_r = 120 psf
W_i = 120 psf
Ka = .162

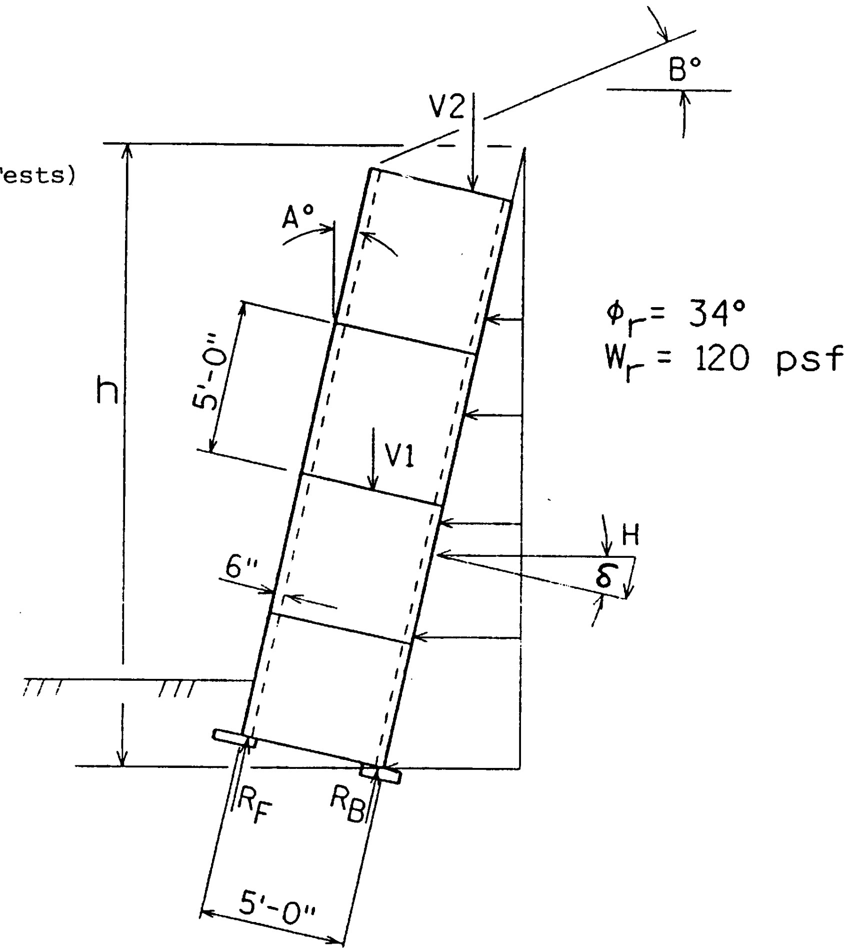


FIGURE 14.27

Modular size = 5'x 5' x 5' with 6" walls and no floors.
Weight of module = 5*(5*1+4*1)*.15 = 6.75 kips/module.
Weight of infill = 16*5*.12 = 9.6 kips/module.
δ = 1/2 ϕ_r = 17° (AASHTO 5.9.2)
H = 1/2 KaW_rh^2
h = 20*cosA + [5./(cos(A+B))]*sinB + 5*sinA
h = 18.79 + 1.58 + 1.71 = 22.08 ft.
H = 1/2*.162*.120*22.08^2 = 4.74 kips
V1 conc. = 4*6.75/5 = 5.4 kips/ft. of wall
V1 soil = 4*9.6/5 = 7.68 kips/ft. of wall
V2 = 5*5*(tan(A+B))*.5*.12 = 1.05 kips/ft. of wall

Determine factor of safety against overturning

Note: System was full scale tested to validate that 80% of infill weight is carried by the walls.

Take moments about toe.
Overturning Moment = H cosδ*((h/cosA)/3.) = 4.74*.956*23.5/3 = 35.5 ft. kips.

Resisting Moment = Hsind*5 + V1conc*(10*sinA+2.5*cosA) + V1soil *.8*(10*sinA+2.5*cosA) + V2*.8*(20*sinA+2/3*5*cosA)
= 1.39*5+5.4*(3.42+2.35)
+ 7.68*.8*5.77 + 1.05*.8*(6.84+3.13) = 81.93 ft.-kips

FS overturning = Resisting Moment/Overturning Moment = 81.93/35.5 = 2.3
Minimum required = 2.0, OK

Determine footing reactions

Take summation of moments about the center of each wall to determine reaction parallel to each wall. Neglect 20% of soil weight.

Force components parallel to wall:

V1 conc*cosA = 5.07 kips
80% V1 soil*cosA = 5.77 kips
80% V2*cosA = .79 kips
Hsinδ = 1.39 kips

Force components perpendicular to wall:

V1 conc*sinA = 1.85 kips
80% V1 soil*sinA = 2.10 kips
80% V2 *sinA = .29 kips
H cosδ = 4.53 kips

Moment summation about center of front wall:

R_B*4.5=5.07*2.25+5.77*2.25+.79*3.08+1.39*4.75
+1.85*10+2.10*10+.29*20-4.53*7.83
R_B*4.5=11.41+12.98+2.43+6.60+18.5+21.0+5.8-35.47
R_B = 43.25/4.5=9.61 kips/ft. of wall

Moment summation about center of back wall:

R_F*4.5=5.07*2.25+5.77*2.25+.79*1.42-1.39*.25
-1.85*10-2.10*10-.29*20+4.53*7.83
R_F*4.5=11.41+12.98+1.12-.35-18.5-21.0-5.8+35.47
R_F=15.33/4.5=3.41 kips/ft. of wall
R_B+R_F = 13.02 kips/ft. of wall=S forces parallel to wall

The bearing pressure can now be found below the footings and must be less than or equal to the allowable bearing capacity.

<u>Determine factor of safety against sliding</u>

Neglect passive resistance at toe
Assume coefficient of sliding friction between concrete footing and insitu soil = .5
Assume coefficient of sliding friction between infill and insitu soil = .67
Assume 80% of infill is supported by footings and 20% by insitu soil

The force producing sliding equals the summation of force components perpendicular to the wall. (see calculations for "Determine footing reactions")
4.53-1.85-2.10-.29 = 0.29 kips

The force resisting sliding equals the summation of force components parallel to the wall times the coefficient of sliding friction.

.5(5.07+5.77+.79+1.39)+.67*.2/.8(5.77+.79) = 6.51 + 1.10 = 7.61 kips
FS sliding=Force Resisting Sliding/Force Producing Sliding = 7.61/.29 = 26
Minimum required = 1.5 , OK

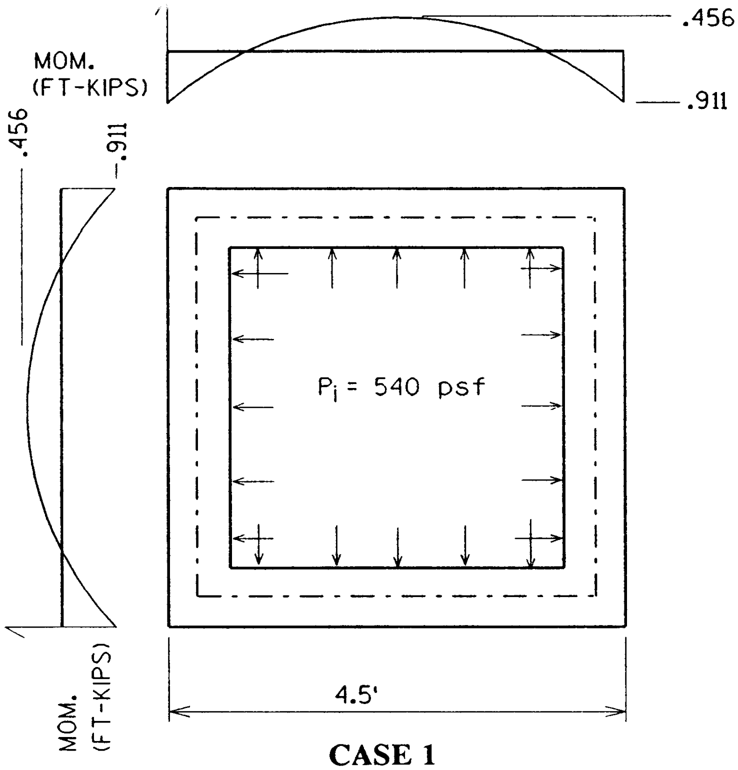
<u>Determine area of steel in bin walls to resist internal bin pressure</u>

Pi = maximum pressure inside bin
Pi = wb = .120*4.5=.54 kips/ft^2
Px = maximum pressure outside bin
Px = KaW_rh = .162*.12*22.08=.429 kips/ft^2

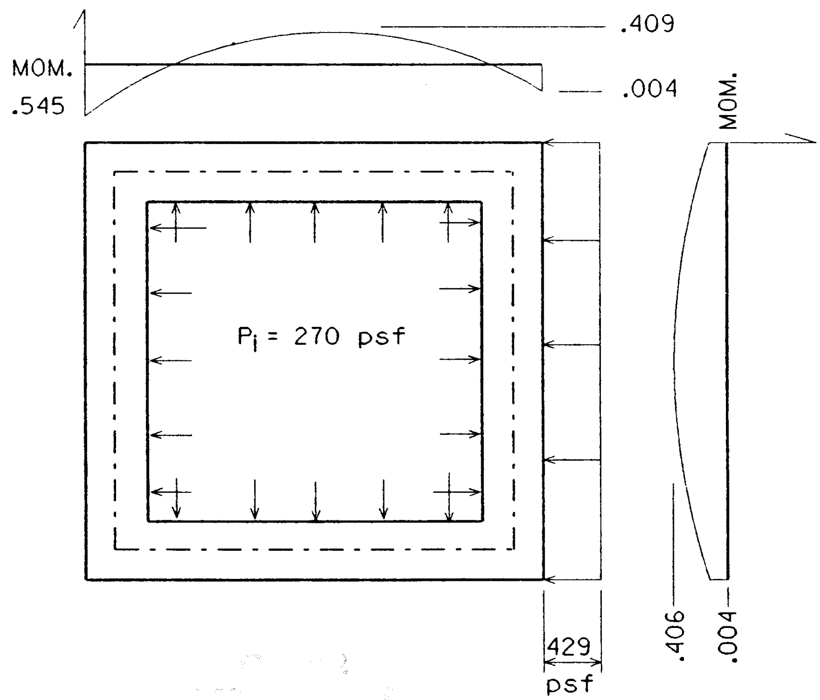
The bin walls must be analyzed for two loading cases.

Case 1 - An internal bin pressure of 540 psf on all four walls.
Case 2 - An internal bin pressure of 270 psf on all four walls plus an external pressure of 429 psf on the back wall only.

Using "ICES STRUDL" to analyze a box for Case 1 the corner moments are +.911 ft kips and the moments in the center of the box are -.456 ft. kips. For Case 2 the maximum corner moment (which occurs at the front face) are + .545 ft. kips and the moment at the center of the box at the back face is + .406 ft. kips. The axial and shear force for Case 1 is 1.215 kips. Figure 14.28 shows the moment diagrams from Case 1 and 2 loading on a particular type of bin wall. Bin wall modules may have different shapes and sizes.



CASE 1



CASE 2

FIGURE 14.28

MOMENT DIAGRAMS FROM LATERAL PRESSURE ACTING ON ONE TYPE OF BIN WALL

Assuming all of the reinforcing steel is placed in the center of the 6 inch wall, the area of steel required for bending based on the maximum corner moment of .911 ft. kips and a load factor of 1.69 is:

$R = 1.69*M/\phi bd^2=1.69*.911/(.9*3^2) = .190$ k.s.i.

Using design tables the percentage of steel = .0033

$As = .0033*12*3 = .1188\ in^2/ft.$

The area of steel required to carry the axial force of 1.215 kips =
$1.215*1.69/(.9*60)=.038in^2/ft.$ total steel required = $.157\ in^2/ft$

<u>Determine bearing stress between bottom of lower module and top of footing</u>

The reaction from the back wall (R_B) is 9.61 kips/ft. of wall (See Figure 14.27)
The contact area between footing and lower module is equal to the wall thickness which is 6" times a 1 foot width of wall.
actual stress = 9.6/(12*6) = .133 kips/in^2
allowable stress = .3*3.5 = 1.05 k.s.i. OK (where f'c of footing = 3.5 ksi)

(3) Summary of Design Safety Factors and Requirements

Requirements

a. Safety Factors

Overturning $\geq$ 2.0
Sliding $\geq$ 1.5
Global $\geq$ 1.3

b. Foundation Design Parameters

Use values provided by WisDOT

c. Concrete Design Data

f'c = 4000 psi (or as required by design)
fy = 60,000 psi
Use uncoated bars or welded wire fabric

d. Traffic Surcharge

Traffic live load surcharge = 2 feet = 240 lb/ft^2

e. Retained Soil

Unit weight = 120 lb/ft^3
Angle of internal friction
Use value provided by WisDOT.

f. Soil Pressure Theory

| Coulomb's Theory.

14.3(J) MSE WIRE FACED WALLS

MSE wire faced walls are essentially MSE walls with a welded wire fabric facing instead of a precast concrete facing. The internal and external analysis is identical to that explained in 14.3(G)(1). A theoretical stress analysis of the welded wire fabric facing is usually not performed by suppliers of these systems and is not required. The wire size, spacing and patterns used in the facing are developed from performance data of full size wall tests and from applications in actual walls. A test to determine the connection strength between the soil reinforcement and the facing panels is required. Some systems do not use a connection because the ground reinforcement and facing panel are of one piece construction. MSE wire faced wall systems usually incorporate a backing mat behind the front facing. A fine metallic screen or filter fabric is placed behind the backing mat (or behind the facing if a backing mat is not used) to prevent the backfill from passing thru the front face. MSE wire faced walls can tolerate considerable differential settlement because of the flexibility of the wire facing. The limiting differential settlement is 1/50. The flexibility of the wire facing results in face bulging between ground reinforcement. The actual amount varies per system but normally is under one inch. Recommended limits on bulging are 2" for permanent walls and 3" for temporary walls.

When MSE wire faced walls are used for permanent wall applications, all steel components must be galvanized. When used for temporary wall applications black steel (non-galvanized) may be used since the walls are usually left in place and buried. The service life of a temporary wall is generally 3 years.

Temporary MSE wire faced walls can be used as a replacement for temporary sheeting and shoring if site conditions permit. They can also be used when staged construction is required to maintain traffic when an existing roadway is being raised and/or widened in conjunction with bridge approaches, railroad crossings or road reconstruction.

(1) Design Procedure for MSE Wire Faced Walls

The design procedure for MSE Wire Faced Walls is identical to the design procedure for "MSE Vertical Face Walls with Metallic (Inextensible) Soil Reinforcement" as described in this manual except as noted below. See Section 14.3(G)1 of this manual for a design example.

The force in the soil reinforcement at the face of the panel cannot exceed 0.5 of the connection strength failure load determined from tests. (Minimum factor of safety of 2.0). WisDOT has the option to waive connection strength tests for systems which use one component for soil reinforcement and facing. Because MSE Wire Faced Walls may undergo relative deformation in the 3 inch range at failure loads without detrimental performance, setting limits on deformation is not relevant. Wire facing panels carry their loads much like a cable subject to a uniform load. A cable carrying a uniform load assumes the form of a parabola and the tension force in the cable can be estimated from the equation $wL^2/8d$, or the more exact equation $wL/2*SQRT\ (1 + L^2/16d^2)$ where w is the uniform load, L is the distance between supports and d is the sag in the cable. By measuring the bulging or sag the tension force in the welded wire fabric facing can be determined.

The welded wire fabric used to fabricate wire faced walls shall have a yield stress of 65,000 psi. Permanent wire faced walls are designed for a 75 year life. Temporary walls are designed for a 3 year life. The maximum allowable stress in the reduced section of wire faced walls after sacrificial steel has been removed at the end of the design life is .47 Fy. All steel shall be shop fabricated of cold drawn steel wire conforming to the minimum requirements of ASTM A-82 and welded into the finished configuration in accordance with ASTM A-185. Walls used in a permanent application shall have all steel galvanized to conform to the minimum requirements of ASTM A-123.

The minimum embedment (toe bury depth) of MSE wire faced walls shall be as stated in AASHTO 5.8.1 except frost depth shall not be considered. The material requirements of the backfill in the reinforced soil zone are identical to the requirements for MSE walls with precast concrete panel facings. The overall vertical tolerance of the wall and the horizontal alignment tolerance shall not exceed 2" per 10 feet for permanent installations and 3" per 10 feet for temporary installations.

(2) Summary of Design Safety Factors and Requirements

a. Safety factors

Overturning	≥	2.0
Sliding	≥	1.5
Pullout	≥	1.5
Connection Strength	≥	2.0
Global	≥	1.3

b. Welded Wire Fabric

fy = 65,000 psi (specifying yield strengths above 65,000 psi is not allowed)

fs = 30,550 psi

ASTM Specifications A82, A185, A123

c. Traffic Surcharge

Traffic live load surcharge = 2 feet = 240 lb/ft^2

d. Retained Soil

Unit Weight = 120 lb/ft^2

Angle of internal friction as determined from tests from WisDOT.

e. Design Life

75 year minimum for permanent walls
3 year minimum for temporary walls

f. Soil Pressure Theory

Coulomb's Theory.

g. Soil Reinforcement

Soil reinforcement shall be grid steel systems. The minimum soil reinforcement length shall be 70 percent of the wall height and not less than 6 feet. The length of soil reinforcement shall be equal from top to bottom. Soil reinforcement must extend a minimum of 3 feet beyond the failure plane.

14.3(K) MSE WALLS WITH CAST IN PLACE FACING

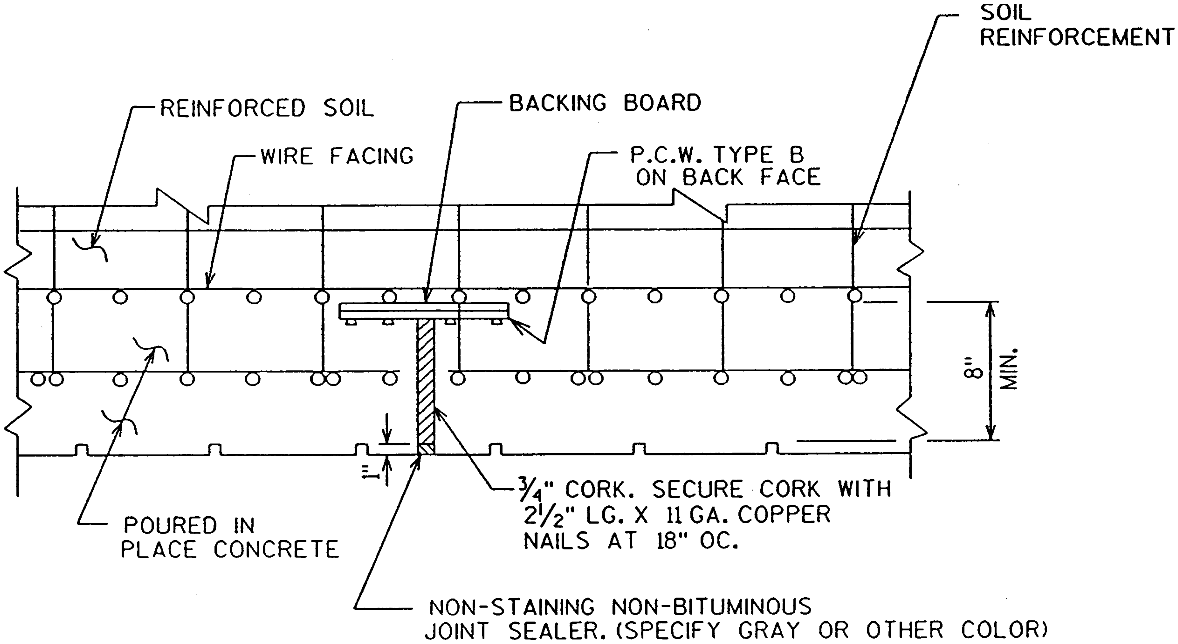
MSE walls with cast in place concrete facings are identical to MSE wire faced walls except a cast in place concrete facing is added after the wire face wall is erected. Modifications are made to the standard wire face wall detail to anchor the concrete facing to the wire facing and soil reinforcement. They are usually used when a special aesthetic facial treatment is required without the numerous joints that are common to precast panels. They can also be used where differential settlement is above tolerable limits for other wall types. A MSE wire faced wall can be constructed and allowed to settle with the concrete facing added after consolidation of the foundation soils has occurred.

The cast in place concrete facing shall be a minimum of 8 inches thick and contain coated or galvanized reinforcing steel. This is required because the panels and/or anchor that extend into the cast in place concrete are galvanized and a corrosion cell would be created if black steel contacts galvanized steel. All wire ties and bar chairs used in the cast in place concrete must also be coated or galvanized. Note that the 8 inch minimum wall thickness will occur at the points of maximum panel bulging and that the wall will be thicker at other locations. Also note that the 8 inch minimum is measured from the trough of any formliner or rustication.

Vertical construction joints are required in the cast in place concrete facing to allow for expansion and contraction and to allow for some differential settlement. Closer spacing of vertical construction joints is required when differential settlement may occur, but by delaying the placement of the cast in place concrete the effects of differential settlement is minimized. Higher walls also require closer spacing of vertical construction joint if differential settlement is anticipated. Horizontal construction joints may disrupt the flow of a special aesthetic facial treatment and are sometimes not allowed for that reason. The designer should specify if optional horizontal construction joints are allowed. Cork filler is placed at vertical construction joints because cork is compressible and will allow some expansion and rotation to occur at the joint. An expandable polyvinyl chloride waterstop (P.C.W.) is used on the back side of a vertical construction joint. Since forms are only used at the front face of the wall the PCW can be attached to a 10 inch board which is supported by the wire facing. The 8 inch minimum wall thickness may be decreased at the location of the vertical construction joint to accommodate the PCW and its support board. See Figure 14.28a for vertical construction joint detail.

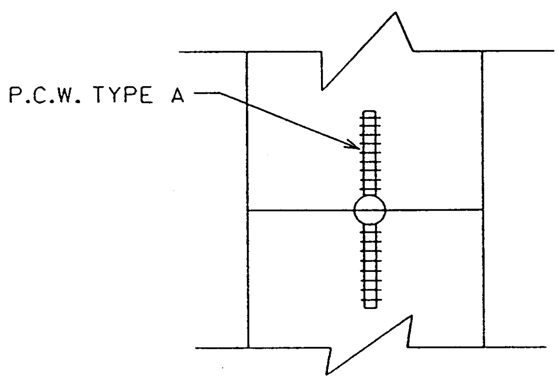
(1) Design Procedure for MSE Walls with Cast In Place Facing

The design procedures for MSE Walls with Cast In Place Facing is identical to the design procedure for MSE Wire Faced Walls as described in Section 14.3(J) of this manual. The required area of reinforcing steel within the cast in place face must be based on an analysis considering the lateral earth pressure acting on the facing and the reactions from the soil reinforcement connections. The influence of wire facing which is outside of the cast in place facing is ignored. The minimum area of steel shall be based on shrinkage and temperature requirements as stated in AASHTO 8.20. Minimum concrete cover shall be 2 inches. The minimum 28 day compressive strength of the concrete shall be 3500 psi. Reinforcing steel may be bars or welded wire fabric in one layer placed near the middle of the wall or in two layers, each layer 2 inches clear from the front face and back face.



VERTICAL JOINT DETAIL

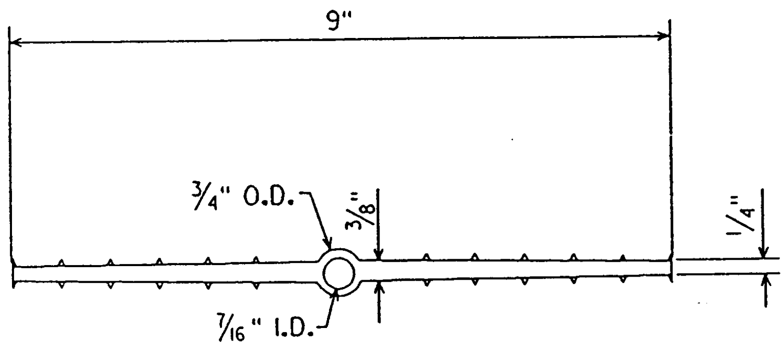
MAX. SPACING OF JOINT = 25'-0" FOR WALLS UP TO 15 FEET HIGH AND 20'-0" FOR WALLS OVER 15 FEET HIGH.



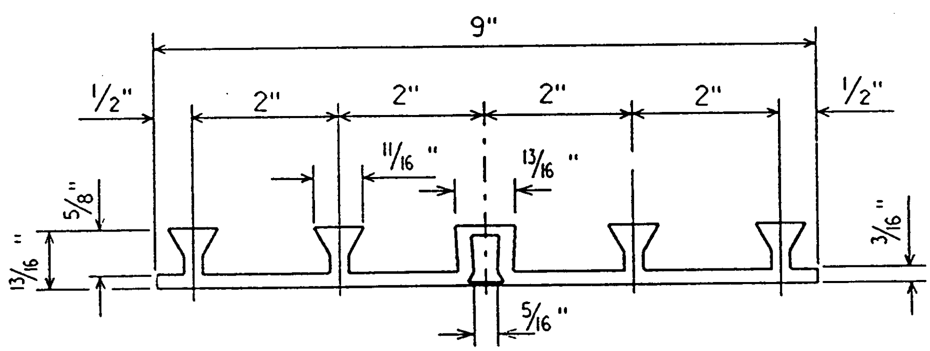
HORIZ. JOINT DETAIL

FIGURE 14.28A

JOINT DETAILS FOR MSE WALLS WITH CAST-IN-PLACE FACING



P.C.W. "TYPE A"
(POLYVINYL CHLORIDE WATERSTOP)



P.C.W. "TYPE B"
(POLYVINYL CHLORIDE WATERSTOP)

FIGURE 14.28A CON'T.

(2) Summary of Design Safety Factors and Requirements

a. Safety factors

Overturning	≥	2.0
Sliding	≥	1.5
Pullout	≥	1.5
Connection Strength	≥	2.0
Soil Reinforcement to Wire Facing	≥	2.0
Wire Facing to C.I.P. Concrete	≥	1.5
Global	≥	1.3

b. Welded Wire Fabric

fy = 65,000 psi (specifying yield strengths above 65,000 psi is not allowed)

fs = 30,550 psi

ASTM Specifications A82, A185, A123

c. Traffic Surcharge

Traffic live load surcharge = 2 feet = 240 lb/ft^2

d. Retained Soil

Unit Weight = 120 lb/ft^2

Angle of internal friction as determined from tests from WisDOT.

e. Design Life

75 year minimum

f. Soil Pressure Theory

| Coulomb's Theory.

g. Soil Reinforcement

Soil reinforcement shall be grid steel systems. The minimum soil reinforcement length shall be 70 percent of the wall height and not less than 6 feet. The length of soil reinforcement shall be equal from top to bottom. Soil reinforcement must extend a minimum of 3 feet beyond the failure plane.

h. Concrete Facings

f'c = 3500 psi

fy = 60,000 psi (bars)

fy = 65,000 psi (welded wire fabric)

Min. thickness = 8.0 inches

Min. reinforcement = 1/8 square inch per foot in each direction (galvanized or coated)

Min. concrete cover = 2.0 inches

14.4 CONTRACT PLAN REQUIREMENTS

The following minimum information shall be required on the plans.

1. Finish grades at rear and front of wall at 25 foot intervals or less.
2. Final cross sections as required for wall designer.
3. Beginning and end stations of wall and offsets from reference line to front face of walls. If reference line is a horizontal curve give offsets from a tangent to the curve.
4. Location of right-of-way boundaries, and construction easements relative to the front face of the walls.
5. Location of utilities if any and indicate whether to remain in place or be relocated or abandoned.
6. Special requirements on top of wall such as copings, railings, or traffic barriers.
7. Footing or leveling pad elevations if different than standard.
8. General notes on standard insert sheets.
9. Soil design parameters for retained soil, backfill soil and foundation soil including angle of internal friction, cohesion, coefficient of sliding friction, groundwater information and ultimate and/or allowable bearing capacity for foundation soil. If piles are required, give skin friction values and end bearing values for displacement piles and/or the allowable steel stress and anticipated driving elevation for end bearing piles.
10. Soil borings.
11. Details of special architectural treatment required for each wall system.
12. Wall systems, system or sub-systems allowed on projects.
13. Abutment details if wall is component of an abutment.
14. Connection and/or joint details where wall joins another structure.
15. Groundwater elevations.
16. Drainage provisions at heel of wall foundations.
17. Drainage at top of wall to divert run-off water.

14.5 CONSTRUCTION DOCUMENTS

(1) Bid Items and Method of Measurement

Proprietary Retaining Walls shall include all required bid items necessary to build the wall system provided by the contractor. The unit of measurement shall be square feet and shall include the exposed wall area between the footing and the top of the wall measured to the top of any copings. For setback walls the area shall be based on the walls projection on a vertical plane. The bid item includes designing the wall preparing plans, furnishing and placing all materials, including all excavations, temporary bracing, piling, (including delivered and driven), poured in place or precast concrete or blocks, leveling pads, soil reinforcement systems, structural steel, reinforcing steel, backfills and infills, drainage systems and aggregate, geotextiles, architectural treatment including painting and/or staining, drilled shafts, wall toppings unless excluded by contract, wall plantings, joint fillers, and also all labor, tools, equipment and incidentals necessary to complete the work.

The contractor will be paid for the plan quantity as shown on the plans. (The intent is a lump sum bid item but is bid as square feet of wall). The top of wall coping is any type of cap placed on the wall. It does not include any barriers. Measurement is to the bottom of the barrier when computing exposed wall area.

Non-proprietary Retaining Walls are bid based on the quantity of materials used to construct the wall such as concrete, reinforcing steel, piling, etc. These walls are:

Cast-in-Place Concrete Cantilever Walls
Post and Panel Walls
Steel Sheet Piling Walls

(2) Special Provisions

The Structures Design Section has Standard Special Provisions for:

Mechanically Stabilized Earth Modular Block Walls, Item 90031
Mechanically Stabilized Earth Concrete Panel Wall, Item 90031
Mechanically Stabilized Earth Walls with Cast-in-Place Facing, Item 90031
Mechanically Stabilized Earth Wire Faced Wall, Item 90031
Modular Block Gravity Wall, Item 90031
Modular Bin or Crib Wall, Item 90031
Gabion Wall, Item 90034

The designer determines what wall systems/system are applicable for the project. The approved names of suppliers are inserted for each eligible wall system. The list of approved proprietary wall suppliers is maintained by the Structures Development Section which is responsible for the Approval Process for earth retaining walls, Appendix A.

14.6 WALL AESTHETICS

(1) Introduction

Retaining walls are one of the key road design elements. Along with the direct function of holding back earth, they provide opportunities for aesthetic enhancement of transportation systems. Retaining walls act as a link between various highway structures and adjacent land forms. Additionally, when multiple walls exist along a corridor, repetition of a similar design will provide continuity throughout that corridor. Therefore, the designer must be aware of the total impact of retaining walls within the roadway corridor and determine how to treat them aesthetically so that they blend into their surrounding environment. The designer should be conscious of the traveler's view of the wall as well as the view of those adjacent to the corridor.

Public Input

The aesthetic elements surrounding any particular retaining wall are key to public acceptance of a wall project. Early in the wall design process the designer should review any comments about the wall generated from public meetings during the preliminary design and environmental documentation process. Where possible these comments should be considered in the design.

Overview of Retaining Wall Aesthetic Design

There is often uncertainty associated with aesthetics and there is no universally accepted theory. Simply defined, **aesthetic qualities are the visual qualities that contribute to a perception of well-being and quality of life as defined by a cross-section of society.**

Because of the uncertainties surrounding aesthetics, and the lack of any universally accepted theory of aesthetics, the following three step process has been established to help the designer to fully consider the aesthetic elements of retaining walls. These steps need to be integrated and considered together, not as discrete individual actions. Aesthetics is a careful blending and balancing of materials (wood, concrete or steel), with design elements (line, form, color and texture) and architectural elements (wall caps, parapets, fencing etc.).

(2) Guidelines

STEP 1: Determine whether to involve a landscape architect. Consideration should be given to involving a landscape architect whenever:

- The wall will exceed ten feet in height.

- Extenuating circumstances are present, regardless of wall height. For example, in a rural area the public may request special aesthetic treatments to enhance a scenic area. Other examples include, but are not limited to: historic areas, tourist areas, or other public requests.

The Landscape Architect can provide important information, guidance and early assistance with aesthetic considerations. Involving the Landscape Architect in the design process for a retaining wall will not only result in a more aesthetically pleasing design, but can also result in cost-savings. It is easier and more cost-effective to determine the real costs of a design early rather than requiring expensive add-ons later.

STEP 2: Determine if wall will be placed in an urban or rural setting:

- An urban setting would be one generally dominated by structures with a variety of colors, textures and architectural styles. In addition, the surrounding landscape is often more orderly and manicured, and involves incorporated areas.

- A rural setting is more natural, may include agricultural or forested areas, and generally involves unincorporated areas.

URBAN

In general, more attention should be given to the aesthetic treatment of walls placed in urban areas. The high volume of users, as well as adjacent land owners, who view such structures are increasingly demanding that these walls be given aesthetic treatment so as to reduce any negative visual impacts that may result. The following chart provides general guidance regarding various aspects of retaining wall design for urban areas.

GENERAL AESTHETIC GUIDELINES FOR RETAINING WALLS IN URBAN AREAS

	URBAN
LINE AND FORM	curvilinear (cords of a wall that reflect adjacent landforms)
COLOR	respond to local setting using: colored concrete block, concrete stain or paint; natural or stained wood; painted or self weathering steel or dip treatment to turn steel dull or gray
TEXTURE	respond to local setting; could be highly textured, e.g. stone, barnboard concrete, bushammered concrete, ribbed, form liner, cribbing, or other rustication; binwall with textured concrete veneer
LEVEL OF DETAILING	may be highly detailed
ACCENTS	may include pilasters, arches and caps; contrasting lines, colors and textures
FENCING	may include custom design

RURAL

The amount of aesthetic treatment given to walls which are placed in rural settings is dependent upon further classification. Rural highways can be placed into two categories, Regular Highways and Special Highways.

- Regular Highways are those roads which generally carry high levels of commercial traffic, medium levels of commuter traffic, and low levels of tourist traffic. These roadways may be either 2 or 4 lane divided or undivided highways. These roads require only standard aesthetic treatment.

- Special Highways are those roads which carry high levels of tourist traffic, and medium to low levels of commercial and commuter traffic. Additionally, these roads are highly scenic, and pass through, link or are adjacent to parks, tourist areas, recreational areas or historic areas. These roadways may be either 2 or 4 lane divided or undivided highways. High priority will be given to the recreational driving experience and aesthetic treatment.

The following chart gives general guidance regarding various aspects of retaining wall design for Regular Highways and Special Highways.

GENERAL AESTHETIC GUIDELINES FOR RETAINING WALLS IN RURAL AREAS

	Special Highways	Regular Highways
LINE AND FORM	curvilinear (cords of a wall that reflect adjacent landforms)	angular
COLOR	respond to local setting using: colored concrete block, concrete stain or paint; natural or stained wood; painted or self weathering steel or dip treatment to turn steel dull or gray	economical response to local setting using: colored concrete block, natural stained or painted concrete, natural, stained, or painted wood; self weathering steel
TEXTURE	respond to local setting; could be highly textured, e.g. stone, barnboard concrete, bushammered concrete, ribbed, form liner, cribbing, or other rustication; binwall with textured concrete veneer	economical match to local setting e.g. rubbed finish concrete, sand blasted concrete
LEVEL OF DETAILING	may be highly detailed	simple detailing
ACCENTS	may include pilasters, arches and caps; contrasting lines, color and textures	minimal accents
FENCING	may include custom design	simple design

Finally, the designer should consider other factors that may have an impact on the aesthetic elements of the design.

STEP 3: Other Factors to Consider

- Walls should not dominate the area of effective vision of the driver
- Use the wall to mount necessary light fixtures

- Extend walls to meet overpasses and bridge abutments
- Wall elevation should follow the natural grade of the land
- Taper the ends of walls to meet adjacent slopes
- Align walls to follow adjacent landforms or as required by roadway alignments
- When possible, vary wall alignments
- Backfill slopes should not exceed 2:1 for revegetation
- Provide drainage at the base of upslope walls
- For consistency, coordinate and repeat fixtures, wall finishes, patterns, line, form, color and texture to emphasize continuity for the entire transportation system within a given locality

Retaining wall aesthetics will be further addressed by the FDM Landscape and Aesthetic Design Chapter later this year. Central Office Design Landscape Architects are available to assist designers.

14.7 PLANTING FOR LIVING WALL SYSTEMS

(1) Introduction

Living wall systems can offer unique opportunities to enhance the character and aesthetics of a transportation corridor. The integration of vegetation into a wall structure not only allows that structure to blend into the surrounding landscape, thus de-emphasizing its visual impact, but can also provide enhanced visual interest with the use of vegetative species which display highly appealing flowers or foliage.

When considering the use of a living wall, the designer should be aware that harsher growing conditions are inherent in such systems, therefore plant survivability will depend on the proper selection of plant materials and the application of various implementation and maintenance strategies.

Plant Selection

Living wall systems consist of containers or planters located above ground, which results in unique freeze/thaw dynamics. In these systems, plant roots are removed from the ground environment and its warming influence, as a result earlier and heavier freezing may be the norm. In order for the plants to tolerate these conditions, they should be selected from one Zone north of the USDA Plant Hardiness Zone in which the system is being constructed. Plant hardiness refers to a plant's ability to survive over winter, thus selecting plants that would be more winter hardy may improve the chances for successful and healthy growth.

Drought conditions may be common with these systems as there may be high evaporation and low moisture infiltration. The plant species selected should be able to tolerate drought conditions.

Due to the fact that these systems will be constructed along the highway system, they will be highly susceptible to aerial salt spray. Salt spray can severely damage or kill certain species so it is important to select those species that have a high tolerance to salt spray.

The soil pockets will better accommodate medium to small growing shrubs, vines and groundcovers. The soil pockets in these systems are usually not deep enough to sustain large shrub species as the root systems cannot properly anchor themselves.

As there is a growing public awareness and demand for aesthetically appealing environments, selecting vegetation that is appropriate to the setting, native to the area, and adds seasonal interest is important. Vines and groundcovers may be selected so as to more effectively cover the structure and make the wall structure visually more attractive. In general, regardless of the species used, in order to more quickly cover the structure the plant spacing should be closer than typically recommended for the particular species.

It is important to note, however, that plant survivability should remain the driving force when selecting vegetation. Plants chosen for their aesthetic appeal may not tolerate the harsh conditions and may soon become unhealthy or die out, thus becoming aesthetically displeasing.

(2) Implementation

The soil in these systems is typically highly altered from any parent material. The soil structure, moisture holding capacity and nutrient levels may not be conducive to healthy plant growth. Adding various soil amendments may improve the quality of the soil.

Drought conditions may be common with these systems as there may be high evaporation and low moisture infiltration. The designer may also consider using a copolymer soil amendment in these systems. The copolymer increases the amount of moisture the soil can hold as well as allowing the soil to retain the moisture for longer periods of time.

Maintenance

Drought conditions may be common with these systems as there may be high evaporation and low moisture infiltration. In order to have optimum plant performance the designer should consider irrigating these systems. If irrigation is to be used, then drainage considerations should be incorporated into the structure so as to insure that moisture levels remain within acceptable levels.

Due to the nature of these structures, the vegetation in these systems may be difficult to maintain. Where possible, vegetation types should be selected which require little or no maintenance.

When considering such a system, plant performance is critical and depends on addressing the above mentioned concerns. The C.O. Design Landscape Architects are available for guidance on any of these matters. There is also helpful information contained in Chapter 7 in the Facilities Development Manual.

14.8 NOISE BARRIER WALLS

(1) Wall Contract Process

WisDOT has classified all noise walls (both proprietary and non-proprietary) into three wall systems. All proprietary systems must be pre-approved prior to being considered for use on WisDOT projects. The three noise wall systems that are considered for WisDOT projects include the following:

1. Double-Sided Sound Absorptive Noise Barriers

2. Single-Sided Sound Absorptive Noise Barriers

3. Reflective Noise Barriers

If a wall is required, the designer must determine which wall system or systems are suitable for a given wall location. In some locations all wall systems may be suitable, whereas in other locations some wall systems may not be suitable. Information on aesthetic qualities and special finishes and colors of proprietary systems is available from the manufacturers. Information on approved concrete paints, stains and coatings is also available from the Office of Design, Structural Development Unit. Designers are encouraged to contact the Structural Development Unit (608-266-8494) if they have any questions about the material presented in the Bridge Manual.

The step by step process required to select a suitable wall system or systems for a given wall location is as follows:

Step 1: Investigate Alternatives

Investigate alternatives to walls such as berms, plantings, etc.

Step 2: Geotechnical Analysis

If a wall is required, WisDOT geotechnical personnel shall conduct a soil investigation at the wall location and analyze the samples to determine soil design parameters for the foundation soil. Geotechnical personnel are also responsible for recommending remedial methods of improving soil bearing capacity if required.

Step 3: Evaluate Basic Wall Restrictions

The designer shall examine the list of suitable wall systems included in the geotechnical report for the site and remove any system that does not meet usage restrictions for the site.

Step 4: Determine Suitable Wall Systems

The designer shall further examine the list of suitable wall systems for conformance to other considerations. Refer to Section 14.6 for a discussion on aesthetic considerations.

Step 5: Determine Contract Letting

After the designer has established a list of a suitable wall system or systems the method of contract letting can be determined. The designer has several options based on the contents of the list.

Option 1:

The list contains only non-proprietary systems.

Under Option 1 WisDOT will furnish a complete design for one of the non-proprietary systems.

Option 2:

The list contains proprietary wall systems only or may contain both proprietary and non-proprietary wall systems but the proprietary wall systems are deemed more appropriate than the non-proprietary systems.

Under Option 2 WisDOT will not furnish a design for any wall system. The contractor can build any wall system which is included on the list. The contractor is responsible for providing the complete design of the wall system selected, either by the wall supplier for proprietary walls or by the contractor's engineer for non-proprietary walls. Contract special provisions, if not in the Supplemental Specs., must be included in the contract document for each wall system that is allowed. Under Option 2, at least two and preferable three wall suppliers must have an approved product that can be used at the project site. See FDM Manual (Procedure 19-1-5) for any exceptions.

Option 3:

The list contains proprietary wall systems and non-proprietary wall systems and the non-proprietary systems are deemed equal or more appropriate than the proprietary systems.

Under Option 3 WisDOT will furnish a complete design for one of the non-proprietary systems and list the other allowable wall systems.

Step 6: Prepare Contract Plans

Refer to Section 14.4 for information required on the contract plans for proprietary systems. If a contractor chooses an alternate wall system, the contractor will have to provide the plans for the wall system chosen.

Step 7: Prepare Contract Special Provisions

The Office of Design and District Office has Special Provisions for each wall type and a generic Special Provision to be used for each project. The list of proprietary wall suppliers is maintained by the Materials Quality Assurance Unit.

Complete the generic Special Provision for your project by inserting the list of wall systems allowed and specifying the approved list of suppliers if proprietary wall systems are selected.

Step 8: Submit P.S. and E. (Plans, Specifications and Estimates)

When the plans are completed and all other data is completed, submit the project into the P.S. and E. Process. Note that there is one bid item, square feet of exposed wall, for all wall quantities.

Step 9: Preconstruction Review

The contractor must supply the name of the wall system supplier and pertinent construction data to the project manager. This data must be accepted by the Office of Design, Contract Plans Section before construction may begin. Refer to the Construction and Materials Manual for specific details.

Step 10: Project Monitoring

It is the responsibility of the project manager to verify that the project is constructed with the previously accepted contract proposal. Refer to the Construction and Materials Manual for monitoring material certification, construction procedures and material requirements.

(2) Pre-Approval Process

The purpose of the pre-approval process is to ascertain that a particular proprietary wall system has the capability of being designed and built according to the requirements and specifications of WisDOT. Any unique design requirements that may be required for a particular system are also identified during the pre-approval process. A design of a pre-approved system is acceptable for construction only after WisDOT has verified that the design is in accordance with the design procedures and criteria stated in the Certification Method of Acceptance for Noise Barrier Walls.

In addition to design criteria, suppliers must provide materials testing data and certification results for the required tests for durability, etc. The submittal requirements for the pre-approval process and other related information are available from the Materials Quality Assurance Unit, Madison, Wisconsin.

REFERENCES

1. Report No. FHWA-RD-89-186, "Durability/Corrosion of Reinforced Soil Structures".

2. Task Force 27-AASHTO-AGC-ARTBA (1990) Ground Modification Techniques for Transportation Applications. AASHTO, Washington, D.C.

3. National Concrete Masonry Association, "Design Manual for Segmental Retaining Walls", 2302 Horse Pen Road, Herndon, Virginia 22071-3406.

4. Standard Specifications for Highway Bridges, AASHTO, 444 North Capitol Street, N.W., Suite 249, Washington, D.C. 20001.

APPENDIX A - SUBMITTAL REQUIREMENTS FOR PRE-APPROVAL PROCESS

APPROVAL PROCESS FOR
RETAINING WALL, PERMANENT
by

WISCONSIN DEPARTMENT OF TRANSPORTATION
DIVISION OF HIGHWAYS
MARCH 23, 1995

A. INTRODUCTIONS

The following four wall systems require the supplier or manufacturer to submit to the Division of Highways, Office of Design, Structural Development Section a package that addresses the items specified in paragraph C.

1. Modular Block Gravity Walls
2. MSE Walls with Modular Block Facings
3. MSE Walls with Precast Concrete Panel Facings
4. Modular Concrete Bin or Crib Walls

B. GENERAL REQUIREMENTS

Approval of Retaining Wall Systems allows for use of these systems on Wisconsin Department of Transportation (WisDOT) projects upon the manufacturer's certification that the system as furnished to the contractor (or purchasing agency) complies with the design procedures specified in the WisDOT Bridge Manual. WisDOT projects include: State, County and Municipal Federal Aid and authorized County and Municipal State Aid projects in addition to materials purchased directly by the state.

The manufacturer shall perform all specification tests with qualified personnel and maintain an acceptable quality control program. The manufacturer shall maintain records of all its control testing performed in the production of Retaining Wall Systems. These test records shall be available at all times for examination by the Construction Materials Engineer for Highways or designee. Approval of materials will be contingent upon satisfactory compliance with procedures and material conformance to requirements as verified by source and field samples. Sampling will be performed by personnel during the manufacture of project specific materials.

C. QUALIFYING DATA REQUIRED FOR APPROVAL

Applicants requesting Approval for a specific system shall provide three copies of the documentation showing that they comply with AASHTO "Standard Specifications for Highway Bridges" and WisDOT Standard Specifications for Road and Bridge Construction and the design criteria specified in the WisDOT Bridge Manual.

1. An overview of the system, including system theory.

2. Laboratory and field data supporting the theory.

3. Detailed design procedures, including sample calculations for installations with no surcharge, level surcharge and sloping surcharge.

4. Details of wall elements, analysis of structural elements, factors of safety, estimated life, corrosion design procedure for soil reinforcement elements, procedures for field and laboratory evaluation including instrumentation and special requirements, if any.

5. Sample material and construction control specifications - showing material type, quality, certifications, field testing and placement procedures.

6. A well documented field construction manual describing in detail, and with illustrations where necessary, the step by step construction sequence.

7. Details for mounting a concrete traffic barrier on the wall adjoining both concrete and flexible pavements (if applicable).

8. Pullout data for facing block/geogrid connection and soil pullout data (if applicable).

9. Submission of practical application with photos for all materials, surface textures and colors representative of products being certified.

10. Submission, if requested, to an on-site production process control review, and record keeping review.

11. List of installations including owner name and wall location.

12. Limitations of the System.

The above materials may be submitted at any time but, to be considered for a particular WisDOT project, must be received a minimum of 15 weeks before the letting date for that project to meet the P.S. & E. schedule. The material should be clearly detailed and presented according to the prescribed outline.

After final review and approval of comments with the Structural Development Section, the manufacturer will be approved to begin presenting the system on qualified projects.

D. MAINTENANCE OF APPROVAL STATUS AS A MANUFACTURER

The supplier or manufacturer must request to be reapproved bi-annually. The request shall be in writing and certify that the plant production process control and materials testing and design procedures haven't changed since the last review. The request shall be received within two years of the previous approval or the approval status will be terminated. Upon request for reapproval an on-site review of plant process control and materials testing may be conducted by WisDOT personnel. Travel expenses for trips outside the State of Wisconsin involved with this review will be borne by the manufacturer.

For periodic on-site reviews, access to the plant operations and materials records shall be provided to a representative of the Construction Materials Engineer during normal working hours upon request.

If the supplier or manufacturer introduces a new material, or cross-section, or a new design procedure, into its product line, the new feature must be submitted for approval. If the new feature/features is significantly different from the original product, the new product may be subjected to a complete review for approval.

E. LOSS OF APPROVED STATUS

Approval to deliver the approved system may be withdrawn under the following conditions:

Design Conformance

1. Construction does not follow design procedures.
2. Incorrect design procedures are used on projects.

Materials

1. Inability to consistently supply material meeting specification.
2. Inability to meet test method precision limits for quality control testing.
3. Lack of maintenance of required records.
4. Improper documentation of shipments.
5. Not maintaining an acceptable quality control program.

The decision to remove approval from a manufacturer on a specific system rests with the Construction Materials Engineer for Highways or the State Bridge Engineer for Highways.

APPENDIX B – RETAINING WALLS

There are two types of modular block retaining walls and each type has its own Standardized Special Provision.

1) Wall Modular Block Gravity
2) Wall Modular Block Mechanically Stabilized Earth (MSE)

Maximum Height of Block Gravity Walls

The maximum height of block gravity walls depends on surcharge backslope above wall and properties of retained soil. Do not specify a block gravity wall if your wall height is greater than the following.

BACKSLOPE	SURCHARGE (feet)	HEIGHT (feet) with PHI = 30 degrees or more	HEIGHT (feet) with PHI less than 30 degrees
0	0	4.33	3.67
3:1	0	3.67	3.50
2:1	0	3.00	3.00
0	1.0	3.00	2.50
0	2.0	2.00	1.67

HEIGHT for this Table only is exposed height.

PHI is the angle of internal friction of the retained soil. For wall heights slightly above the maximum with insufficient right of way to construct an MSE block wall contact Structures Development at 608-266-8494 for possible exemptions.

Right of Way (ROW) Required for MSE Block Walls

The reinforced soil zone should be inside the ROW or inside a permanent easement. The depth of the reinforced soil zone (distance from backface of blocks to end of soil reinforcement) depends on wall height, backslope, surcharge, properties of retained soil and properties of material in the reinforced soil zone.

For wall heights without backslope or surcharge the length of the soil reinforcement will be about 0.75 times the wall height. For backslopes of 2:1 or for 2 foot surcharges the length of the soil reinforcement will be about 0.9 times the wall height. If Crushed Limestone, Open Graded #2 is used in the reinforced soil zone the length of the soil reinforcement will be about 0.15 times the wall height, less than the values stated above. The minimum length in all cases is 6'. It can be reduced at utility pole locations.

Temporary Easement for Excavation

The maximum allowable safe slope of the excavation behind the reinforced soil zone is specified by OSHA, Regulations (Standards-29 CFR), Excavations, Sloping and Benching and Excavations, Soil Classifications. The safe slope is based on the soil type being excavated.

SOIL TYPE A – MAX. SLOPE = ¾ H : 1 V (53 deg.)
Soil Type A includes stiff to hard consistency cohesive soil (clay) with an unconfined compressive strength of 1.5 tons per square foot or greater.

SOIL TYPE B – MAX. SLOPE = 1 H : 1 V (45 deg.)
Soil Type B is cohesive soil with an unconfined compressive strength greater than 0.5 tons per square foot but less than 1.5 tsf. Soil Type B is also cohesionless soils including: angular gravel (similar to crushed rock), silt, silt loam, sandy loam and in some cases, silty clay loam and sandy clay loam.

SOIL TYPE C – MAX. SLOPE = 1.5 H : 1 V (34 deg.)
Soil Type C is a cohesive soil with an unconfined compressive strength of 0.5 tsf or less or granular soils including gravel, sand or loamy sand.

OSHA Regulations, Requirements for Protective Systems, 1926.652(b), Option 4, allows the excavation of slopes steeper than the maximum slopes specified above if the slope system is designed by a registered professional engineer. Under this option a Type A soil was excavated with the top of the excavation sloped at ¾ to 1 and the bottom 5.5 feet sloped at 0.1 to 1.

Utility poles that are near the top of cut slopes may need temporary bracing prior to site excavation. Utility poles that are located near the back of block gravity walls may need to be set deeper.

<u>Bridge Manual and Structure Numbers</u>

Additional information on usage restrictions for MSE walls is in the Bridge Manual, Chapter 14, Section 14.3(F)(1). Walls over 5.5 feet in height measured from top of leveling pad to bottom of wall cap require an R structure number. Information on when a geotechnical investigation is required is in Section 14.1(2), Step 4 of the Bridge Manual.